HYDRAIN - INTEGRATED DRAINAGE DESIGN COMPUTER SYSTEM

VOLUME III. HYDRA - STORM DRAINS

for:

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(Metric Conversion Sheet)

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INTRODUCTION

HYDRA is a storm drain and sanitary sewer analysis and design program. This document describes HYDRA and guides the user through the necessary steps toward designing or analyzing stormwater drains and/or sanitary sewer systems. Originally designed and developed in 1975, the HYDRA program ran on mainframe computer systems. The objective of the HYDRA program was to provide hydraulic design engineers a means of accurately, easily and quickly designing and analyzing storm, sanitary or combined collection systems. HYDRA achieved these objectives with a high degree of success, and for this reason, selected for incorporation into the HYDRAIN system. The HYDRAIN version of HYDRA does everything its mainframe-oriented predecessor does, with the same accuracy, ease and quickness. In fact, it offers several improvements over the original version. Furthermore, if it is being used within HYDRAIN there are those advantages associated with HYDRA's ability to interact with other related hydraulic design programs. The user documentation section of this volume describes these interactions in more detail.

In the HYDRA design process, the program will select pipe size, slope and invert elevations if given certain design criteria. Additionally, HYDRA will perform analyses on an existing system of pipes (and/or ditches). When an existing system of pipes is overloaded, HYDRA will indicate suggested flow removal quantities as well as an increased pipe size as an alternative remedy. Additionally, HYDRA can optionally consider the possibility of surcharged systems. The design procedure is not optimized, so alternatives should be examined.

HYDRA requires the creation of an input file, consisting of commands to describe the drainage system. The commands are placed in a logical sequence, usually from higher to lower elevations. It is possible that several command sequences can produce the same result. The input file, established for a particular collection system by the user, is then executed using the HYDRA analysis program.

This volume contains three major sections that provide details on HYDRA's methods and operation. The first section provides the user with an overview of HYDRA features and components. The second section deals specifically with the technical methods used by HYDRA, beginning with a general description of several topics, followed by narratives on methodologies and a discussion of relevant formulas and commands. The topics include: storm flow computed by the Rational method, storm flow calculated using hydrographic methods, and sanitary flow.

The section also discusses conveyance of the water (resulting from the three methods outlined above) when it has entered the system (i.e., a pipe or channel), describes inlet design or analysis computations, discusses the methodologies used in calculating estimated costs, and explains the hydraulic gradeline and pressure flow methodologies. The inlet routines, the hydraulic gradeline routines, and the pressure flow module were adapted to HYDRA from the stand-alone programs, PAVDRAIN, HGL, and PFSM, respectively.[1, 2, 3] Because the pressure flow module is derived from the EXTRAN module of SWMM, some of the pressurized flow discussion replicates the EXTRAN manual.[4]

The third section provides instruction on how to use HYDRA. The section describes the command line input scheme, and discusses the linkages with the HYDRAIN system and other programs. Three appendixes provide examples, commentary, and a listing of commands.

SYSTEM OVERVIEW

This documentation is aimed at providing information to new users (as well as infrequent or "rusty" users) of HYDRA to bring them to a level of ability sufficient for them to use any feature offered by HYDRA. It is not meant to show every possible type of analysis or situation that HYDRA can handle (however the document provides clear examples of several major types of applications).

This section provides an overview of HYDRA by briefly describing its capabilities and structure. The end of this section includes a key to some of the more frequently used terms and concepts. The following sections provide more detailed information to help the user make the most of HYDRA. The user is advised to scan the table of contents of this document to see exactly what this text offers, how it is arranged, and where to turn to for specific information.

CAPABILITIES AND LIMITATIONS

HYDRA operates in two modes: **design** and **analysis**. There are three possible types of systems on which HYDRA can work:

- Storm drain systems.
- Sanitary (sewage) systems.
- Combined (storm and sanitary) sewer systems.

In this documentation, these types are collectively referred to as **storm drain systems** or **sewer systems**, or simply, **systems**.

As implied by the preceding text, HYDRA is made to perform the following tasks:

1) **Analyze a drainage system design given user-supplied specifications**.

2) **"Free design" its own drainage system based on design criteria supplied by the user**.

To meet these broad objectives, HYDRA was necessarily designed to be an extremely flexible and powerful program. The user is warned that care and responsibility should be exercised when using the program as a decision-making tool. *HYDRA is a design aid only and is not a substitute for sound engineering judgment.* This being mentioned, the following is a list of some of HYDRA's more useful features:

- *Cost estimation* - Capabilities that allow for consideration of de-watering, traffic control, sheeting, shrinkage of backfill, costs of borrow, bedding costs, surface restoration, rock excavation, pipe zone costs, etc. HYDRA is also sufficiently flexible to allow cost criteria to be varied for any segment of pipe in a system, if desired. Ground profiles, either upstream or downstream from any specified point along the system, can also be accepted for consideration in cost estimation, if desired.

- *Models storm flow and offers choice of methods* - HYDRA is capable of "generating" storm flow based on the **Rational method** or modeling user-defined hydrographs in a **hydrologic simulation**, at the user's discretion. This may be particularly advantageous for engineers who wish to compare designs or analysis results based on different methods.

- *Models sanitary flow* - HYDRA "generates" sanitary flow based on the traditional "peaking factor" concept.

- *Models drainage systems of any size* - HYDRA has a data handling algorithm especially designed to accept a drainage system of any realistically conceivable design.

- *Infiltration/Inflow analysis* - HYDRA is ideally suited for making these analyses.

- *Planning* - Use HYDRA for determining the most practical alternate choices for unloading an existing overloaded storm drain or wastewater system and for formulating Master Plans to allow for an orderly growth of these systems. The program's features and capabilities should have far-reaching implications for municipal agencies whose existing sewer systems are under stress from rapid population growth and/or changes in land use patterns.

- *Easy data input structure and quick editing capacity* - All data needed to run HYDRA is in one user-supplied input file, simplifying data editing operations. Furthermore, if the program is run from within the HYDRAIN environment, the input file may be modified without leaving the HYDRAIN program by using the built-in editor. (The User Documentation section describes the capabilities of this screen editor and provides instructions for its use.) Time required for data modification and job resubmission is thus minimized, which enables the user to spend more time on his or her own decision analysis.

STRUCTURE OF HYDRA

The structure and organization of the HYDRA program is similar to many other computer programs. The program reads data, analyzes it, and outputs information for the user's review. When the original HYDRA program was developed, a central design criterion was that the program would maximize simplicity in file maintenance and data editing. Unlike many other hydraulic analysis programs, HYDRA requires only a single input data file. This data file is made up of a list of user-supplied **commands** that specify (describe) the system. All internal analysis by HYDRA is performed according to these commands. Once the commands are assembled into a final working data set, they are collectively called a **command string**. (The following section explores these concepts in more detail.) During analysis (program execution), HYDRA checks the command string for proper format and executability. Output is generated according to the user-supplied instructions of the command string and sent to a separate output file which the user may in turn route to either a printer or screen display. If a run aborts prematurely (before intended analysis is completed), appropriate descriptive error messages are added to the output file. There is also a "status report" feature within HYDRA itself that displays (on the user's screen) when each command in the command string is being worked on, in "echo" format. This allows the user to trace program progress.

KEY TO HYDRA TERMS

To use HYDRA, an understanding of how to prepare a program data file is central. For this reason, it is important to have a clear grasp of the more fundamental modeling terms as they are used in this documentation. The more comfortable the user is with the following terms (and their associated concepts), the easier it will be to put this documentation to use.

- *Command* - A three-letter user-supplied "key word" and its associated completed data field that HYDRA recognizes and accepts as input data for performing a specific task. The user selects these commands according to the function(s) that HYDRA is to perform. Each command must be listed (entered) on a separate line of data. These data lines make up the user's input data set, which is collectively referred to as a **command string** (See entry below.) Command names are three-letter "descriptors" (often abbreviations or acronyms) of the tasks that the commands perform. For example, **PDA** is the command name for "**P**ipe **DA**ta," the command that allows for user-provided specifications of pipes within the system. A complete listing and explanation of available commands is provided in appendix C.

- *Command string* - An arrangement of **commands** that describes a given **system**. A command string is the fundamental user-provided data set that allows HYDRA to analyze or design a system. This data set may be edited to adjust for modifications to the system without having to build a new command string from scratch. Commands and command strings are further discussed in the next section.

- *Lateral* - Either a single **link** (see entry below) or a number of links connected in a series. Other laterals may connect with any given lateral, but each lateral is continuous. Laterals can be any length, and there can be any number of links that describe a lateral and any number of laterals within a system. In this documentation, trunks, mains and interceptors are all referred to as laterals.

- *Link* - A segment connecting two **nodes** (synonymous to a connecting drainage or sewer pipe). As it represents a length, it is specified in meters. A link is the smallest unit that can transport a flow, and is the sole building block of **laterals**. The amount of flow in a given link is a constant. The maximum number of links that can be modeled in a single gravity flow or hydraulic gradeline application is 200 and 50 for a pressure-flow application.

- *Node* - A point where storm, sewage or combined flow can be either injected into or removed from the **system**. The maximum number of nodes that can be modeled in a single gravity flow or hydraulic gradeline application is 200 and 50 for a pressure-flow application.

- *System* - Collectively, the entire assemblage of **links** and **nodes** (and thus **laterals**) as defined by the user-supplied **commands** of the **command string**. The **system** in this modeling concept is totally synonymous with a storm drain system, a sanitary system, or a combined sewer system.

This concludes the first section of the HYDRA documentation. The next section will provide a technical overview of the methods and operations found in the analysis program.

TECHNICAL INFORMATION

This section provides technical descriptions of the key methodologies employed by HYDRA to assist users in selecting the appropriate solution technique for a given problem. Included are the methods for generating storm and sanitary flows, flow conveyance, inlet computations, storage, cost estimating, calculating the hydraulic gradeline, and simulating pressure flow.

STORM FLOW: THE RATIONAL METHOD

Developed towards the end of the 19th century, the Rational method is still widely used as a method for computing quantities of stormwater runoff. The Rational method equation is of the form:

$$Q = K \times C \times i \times A \tag{1}$$

Where:

Q	=	The peak flow, m^3/s.
K	=	Constant, 0.00276.
C	=	Runoff coefficient.
i	=	The rainfall intensity, mm/h.
A	=	The area of the watershed, ha.

Intended for determining runoff from <u>small</u>, <u>urban</u> watersheds, use of the Rational method hinges on several basic assumptions:

- The duration used to determine an intensity from an Intensity-Duration- Frequency (IDF) curve is that corresponding to the time it takes for water to flow from the most remote point in the watershed to the point in question, also known as the time of concentration.

- The intensity of the rainfall is constant and is applied to the entire watershed.

- The runoff coefficient remains constant throughout the storm event.

Taking a look at the above assumptions, it becomes clear why this method is intended for small, urban watersheds. To begin, picking an intensity from an IDF curve at a duration equal to the time of concentration makes the most sense in a small, urban environment. Consider, as an ideal case, a large, gently sloping parking lot to be a watershed and apply a rainfall of constant intensity over the entire watershed. It is apparent that the peak flow at the outfall will occur when the entire area is contributing flow; or, to put it another way, when flow from the most remote point in the watershed reaches the outfall. As the watershed characteristics deviate from this ideal case, it becomes more difficult to justify the Rational method because this assumption is likely to be violated. This is particularly true of large rural watersheds.

Another good reason not to apply the Rational method to large watersheds pertains to the second assumption: rainfall is constant throughout the entire watershed. Severe storms, say of a 100-yr return period, generally cover a very small area. Applying the high intensity corresponding to a 100-yr storm to the entire watershed could produce greatly exaggerated flows, as only a fraction of the area may be experiencing such an intensity at any given time.

The variability of the runoff coefficient also favors the application of the Rational method to small, urban watersheds. Although the coefficient is assumed to remain constant, it actually changes during a storm event. The greatest fluctuations take place on unpaved surfaces, as in rural settings. In addition, runoff coefficient values are much more difficult to determine and may not be as accurate for surfaces that are not smooth, uniform, and impervious.

To summarize, the Rational method provides the most reliable results when applied to small, urban watersheds. If it is necessary to apply the method to large or rural areas, then the validity of each assumption should be verified for the site before proceeding.

HYDRA generates storm flows using the Rational method with the **RAI** and **STO** commands (see appendix C for the proper command syntax for these or any other commands mentioned in this documentation). The **RAI** command provides the IDF curve, while the **STO** command provides the balance of the data and triggers the calculations.

To produce flows using the Rational method, HYDRA multiplies the sum of the effective areas (effective area is defined as the product of the area and its respective runoff coefficient, i.e., Σ C×A) by the intensity (from the IDF curve specified in the **RAI** command) corresponding to the longest time of concentration (T_c). To determine the longest T_c, the program examines the origin of all flows entering the junction. The longest time will either be: (1) the T_c specified in one or more **STO** commands directly contributing flow to the junction under analysis, or (2) the sum of the T_c specified by a previous **STO** command and the travel time of that flow through the system to the junction in question. Each time a transport (e.g., **CHA**, **BOX**, **ELP**, **PIP**) command is encountered, HYDRA recalculates both the effective area and the time of concentration.

An exception to this approach occurs when the flow in an individual area exceeds the sum using the long T_c. In situations such as this, the time of concentration corresponding to the area contributing the largest flow is employed, rather than simply using the largest time of concentration. The effective area contributing the lesser flow is then reduced by the ratio of the respective times of concentration. The justification for such a reduction is that in utilizing the smaller T_c, only a fraction of the area corresponding to the greater T_c will contribute flow. This methodology produces greater flows, ultimately resulting in conservative estimates of pipe size. If such a recalculation occurs during a HYDRA run, the user is notified in the output: "**+++ Readjusting sum of C•A**".

HYDRA provides the user with three options for generating time of concentration. These are: (1) user-supplied T_c, (2) overland T_c calculated, gutter T_c supplied by user and (3) both overland and gutter T_c calculated by the program. Should the user desire to supply other values, numerous references (notably FHWA HEC-12 and HEC-19) that will provide theory and guidance in calculating T_c can be consulted.(1, 5) If, however, the user requests that HYDRA calculate the time of concentration, a formula recommended by the Federal Aviation Administration, is used for overland time of concentration.(6)

$$T_c = \frac{3.26 \times (1.1 - C) \times L^{\frac{1}{2}}}{S^{\frac{1}{3}}} \qquad (2)$$

Where:

T_c	=	Time of concentration, min.
C	=	The dimensionless runoff coefficient.
L	=	The distance traveled, m.
S	=	The slope, percent.

If the gutter time is to be calculated, a second formula is used:

$$T_c = \frac{L}{K \times S^{\frac{1}{2}}} \qquad (3)$$

Where:

T_c	=	Time of concentration, s.
L	=	The distance traveled, m.
S	=	The slope, m/m.
K	=	An empirical coefficient equal to 9.81, m/s.

STORM FLOW: HYDROGRAPHIC ANALYSIS

For situations where a system is to be analyzed or designed using hydrographs, HYDRA provides a means of incorporating one or more hydrographs. Through the use of the **UHY** command, a user may introduce a hydrograph to the drainage system. A single hydrograph may be used to represent runoff from multiple areas or different hydrographs may be employed for each land area. This ability is particularly important for analyzing the effects of storage and surcharging in a drainage system.

Several hydrograph generation techniques are available in the HYDRO program which is also a part of the HYDRAIN computer system. Hydrographs generated by HYDRO are automatically placed in the proper format to be read by HYDRA using the **UHY** command. Information about available hydrograph generation techniques can be found in volume II of this series.

SANITARY FLOW

HYDRA allows the user to generate average and peak sanitary flows for both the analysis of existing sewers and the design of new sewers. In addition, the program has the capability for investigating infiltration.

In HYDRA, average sanitary flow is calculated using the parameters specified in one or both of the following combinations of commands: **GPC** and **SAN** or **IPU**, **GPC**, and **SUN**. In the first combination (**GPC**, **SAN**), average flow is calculated by multiplying the number of liters produced per capita per day, (specified in the **GPC** command), by the number of people per hectares times the number of hectares, (specified in the **SAN** command). Therefore, flow in a given conveyance is calculated as follows:

$$Q_a = \sum (uflow \times pop \times area) \tag{4}$$

Where:

Q_a	=	The average flow, using the first combination, L/d.
uflow	=	The unit flow of generated wastewater, L/cap/d.
pop	=	The equivalent population density, cap/ha.
area	=	The area of sanitary collection, ha.

The flow is internally converted to m^3/s. In most circumstances, it will not be necessary to change the value of **uflow**, reducing the equation to:

$$Q_a = uflow \times \sum (pop \times area) \tag{5}$$

In the second combination (**IPU**, **GPC**, **SUN**), average flow is determined by multiplying the cumulative product of number of sanitary units (e.g., houses, apartment buildings, that are specified in the **SUN** command), multiplied by the number of individuals per sanitary unit (specified in the **IPU** command). This total is then multiplied by the number of liters produced per capita per day. This is expressed as:

$$Q_a = uflow \times \sum (units \times ipu) \tag{6}$$

Where:

Q_a	=	The average flow, using the second combination, L/d.
uflow	=	The unit flow of generated wastewater, L/cap/d.
units	=	The number of dwelling units contributing to the system at each node.
ipu	=	The number of people per dwelling unit.

As with the first combination, flow is internally converted to cubic meters per second.

The decision to use the methods discussed above is dependant on the type and availability of data. The **IPU** and **GPC** commands will in most cases only have to be entered once at a point near the beginning of the command string. The **SAN** and **SUN** commands actually introduce flow into the system and are be placed throughout the command string where needed.

In performing calculations to determine peak sanitary flow in a system, HYDRA employs the "peaking factor" concept. The peaking factor is a number, greater than or equal to one, that is multiplied by the average flow to estimate peak loads on the system. In the initial few links of a network, where flows are relatively low, experience has shown that the peaking factor may be as high as 4.0. Further down the line, where the physical length of the system, as well as the increase in flow, makes individual contributions less important, the peaking factor decreases. Since the peaking factor, which is entered in the **PEA** command, can have a significant effect on the amount of flow generated, great care should be taken in selecting the **adf/pf factor** ordered pairs.

Each time one of the transport commands, either **PIP**, **BOX**, **ELP**, or **CHA**, is encountered, the average flow is calculated and a new peaking factor is calculated. At that point, the peak flow in the system is equal to the following:

$$Q_p = pkfctr \times Q_a \tag{7}$$

Where:

Q_p	=	The peak daily flow, L/d.
pkfctr	=	The peaking factor, resulting from the **flow** versus **factor** curve provided in the **PEA** command.
Q_a	=	The average daily flow, L/d.

In addition to accounting for known, anticipated inflows, undesirable inputs, namely infiltration, must also be taken into consideration. Infiltration plays a particularly important role in sanitary sewer systems in that it can comprise a significant percentage of the flow, especially in older networks. Infiltration can be included with the use of the **INF** command. This flow contribution is calculated each time the **SAN** and/or **SUN** command is encountered. It is added to the system flow after the peaking factor has been applied.

FLOW CONVEYANCE

Flow generated in any of the previously described sections is eventually transported through pipes and/or channels. Sizing of these conduits as well as the determination of other flow characteristics can now be accomplished.

HYDRA uses Manning’s formula and the continuity equation to evaluate the adequacy of an existing system to analyze imposed flows or to design a new system. Manning’s formula empirically calculates open channel, gravity induced velocities:

$$V = \frac{1}{n} \times R_h^{\frac{2}{3}} \times S^{\frac{1}{2}} \tag{8}$$

Where:

V	=	The pipe velocity, m/s.
R_h	=	The hydraulic radius of the flow cross section, m.
S	=	The friction slope (assumed to equal the pipe slope), m/m.
n	=	Manning’s friction coefficient for the pipe.

The continuity equation relates flow to velocity and flow area:

$$Q = V \times A \tag{9}$$

Merging equations (8) and (9) yields an equation that empirically calculates flow in an open channel or pipe:

$$Q = \frac{1}{n} \times A \times R_h^{\frac{2}{3}} \times S^{\frac{1}{2}} \tag{10}$$

Where:

Q	=	The pipe flow, m^3/s.
A	=	The area of the flow cross-section, m^2.
R_h	=	The hydraulic radius of the flow cross-section, m.
S	=	The pipe (friction) slope, m/m.
n	=	Manning's friction coefficient for the pipe.

In the design case, this equation is algebraically manipulated to solve for the circular pipe diameter necessary to handle the design flow. HYDRA uses ASTM standard metric sizes in increments of 75 mm for smaller pipes and 150 mm for larger pipes. When designing using English units, circular pipes are between 305 mm and 1220 mm in diameter, and the calculated diameter is rounded up to the nearest 76-mm increment. When the calculated pipe diameter is greater than 1220 mm, it is rounded up to the nearest 152-mm increment. This value, or the minimum diameter value as defined in the **PIP** command, whichever is larger, is then utilized as the design size. It is important to note that even though in certain circumstances design flows may be very low, HYDRA will design no pipe smaller than 305 mm in diameter for storm sewers and 152 mm for sanitary sewers. The **PSZ** command can be used to list a number of pipe diameters to which the pipe design process must be limited.

Since the hydraulic radius is a function of depth and in most cases the circular pipes will not be carrying capacity flows, this term is expressed as follows:

$$R_h = \frac{D}{4} \times \left[1 - \frac{\sin 2\theta}{2\theta}\right] \tag{11}$$

Where:

D	=	The diameter of the circular pipe, m.
θ	=	Theta, in radians, measured as shown in figure 1.

For horizontal elliptical, vertical elliptical, and box pipes (**ELP** and **BOX**, respectively), HYDRA has the capability of designing both the span and the rise. If the user specifies the span, HYDRA designs the rise.

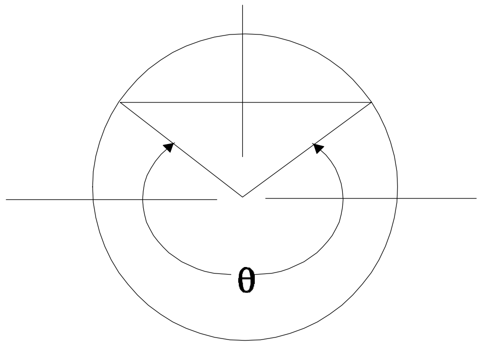

Figure 1. Measurement of θ.[8]

For an open channel, HYDRA presents the user with two alternatives for computing depth and velocity of flow. One alternative, as with calculating flow in pipes, is Manning's equation. Using the design flow and channel geometry, as defined in the **CHA** command, HYDRA employs a trial and error process to determine depth. Once depth is obtained, Manning's equation is solved for velocity.

The second option available for determining depth and velocity of flow in open channels involves the use of a formula developed by Izzard, which is an approximation for hydraulics in gutters.[1]

$$Q = \frac{0.38}{n} \times S_x^{\frac{5}{3}} \times S^{\frac{1}{2}} \times T^{\frac{8}{3}} \tag{12}$$

Where:

S_x	=	The roadway cross slope (vertical to horizontal), m/m.
S	=	The longitudinal slope of the gutter (vertical to horizontal), m/m.
T	=	The spread of flow in the gutter, m.
n	=	Manning's friction coefficient for the gutter.

This equation was derived from Manning's equation to describe flow in wide, shallow, triangular channels. If this option is selected, HYDRA selects the greater of the two side slopes entered and ignores the bottom width. In other words, a channel of the shape depicted in figure 2 is assumed:

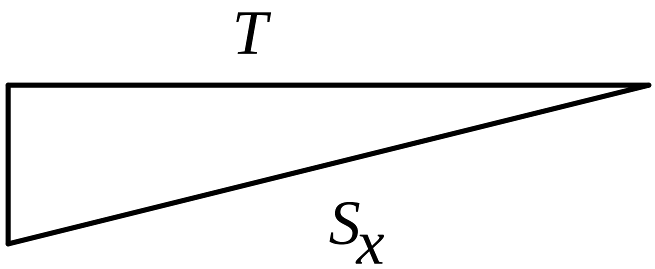

Figure 2. Triangular gutter shape.

The **GUT** command has the same two analysis options as that described for the **CHA** command, and an additional alternative of analyzing composite gutter sections, as shown in figure 3.

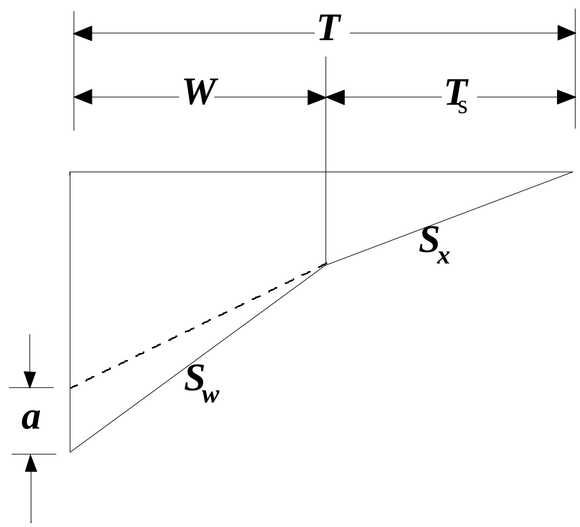

Figure 3. Composite gutter shape.

Equation (12) is also used with this type of gutter section. A trial and error approach is employed, with the width of flow incremented by 3 mm each iteration, until the flow computed in the composite section is equal to the flow produced by HYDRA.

A **GUT** command must precede every **INL** command in order for the inlet to be analyzed or designed correctly. If the gutter option is selected in which the cross slope (S_x) is not entered, then the larger of the two side slope values (larger because the slopes are entered as meters horizontal to meters vertical; so it is actually the flatter slope that is selected) is used as the cross slope.

The **GUT** command, as with the **CHA** command, is available in both hydrographic and steady-state (i.e., Rational method, sanitary, or constant flow) simulations. However, in steady-state simulation runs, the **GUT** command is used only to provide spread, cross slope, velocity, and flow to the **INL** command, whereas, in the hydrographic simulation, the **GUT** command is required.

INLET COMPUTATIONS

Through use of the **INL** command, the user has the ability to design the length or analyze the performance of an inlet given the flows calculated by HYDRA. The capability is available in both hydrographic runs and Rational method peak flow runs, with one important difference. In the hydrographic analysis, the **INL** command is the sole means by which a runoff hydrograph can be introduced into the system. As such, it must appear immediately before one of the system transport commands, either **PIP**, **BOX**, **ELP**, or **CHA**. The inlet size or configuration has a direct influence on system flows. For the Rational method runs, however, the **INL** command serves only to design or analyze an inlet given calculated peak flows. The inlet has no effect on system flows. It is, therefore, an option in steady-state runs.

The basis for the inlet calculations is the HEC-12 manual "Drainage of Highway Pavements."[1] An integral element in the design of pavement drainage is not only the inlet itself, but also the gutter leading to the inlet. The gutter configuration has a direct bearing on the design or performance of the inlet. Therefore, in accordance with the actual pavement drainage process, a gutter command, **GUT**, is required before each **INL** command.

The user has the option of designing and/or analyzing the inlet types listed below and shown in figures 4 through 6:

- Grate inlets.
- Curb inlets.
- Slotted drain inlets.
- User-defined inlets.

Another distinction is made regarding inlet location. Inlet performance is not the same on-grade versus in a sump condition; thus both types of analyses are available. Having outlined the basic uses of the **INL** command, it is necessary to provide a detailed description of the methodologies and equations used for analyzing or designing each of the inlet types.

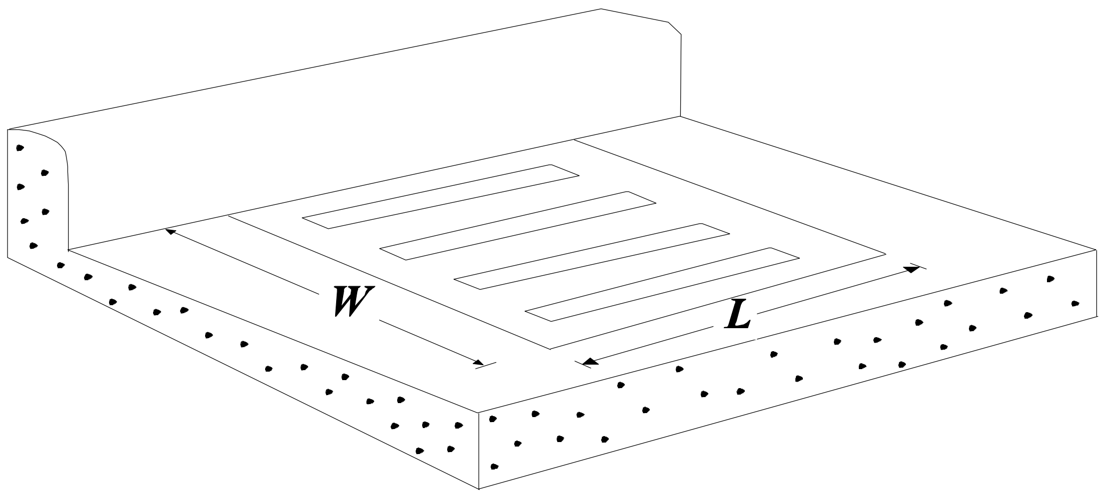

Figure 4. Perspective view of grate inlet.[1]

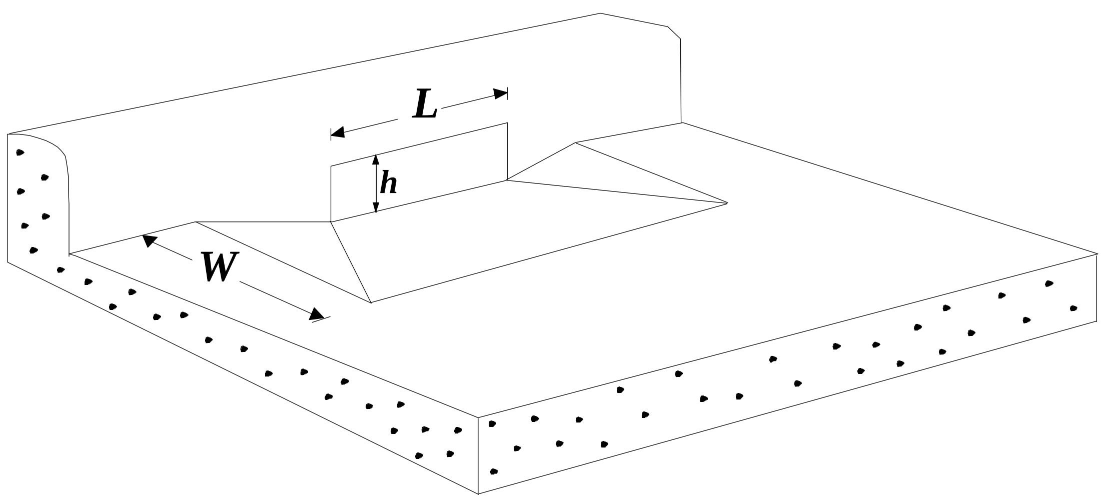

Figure 5. Perspective view of curb inlet.[1]

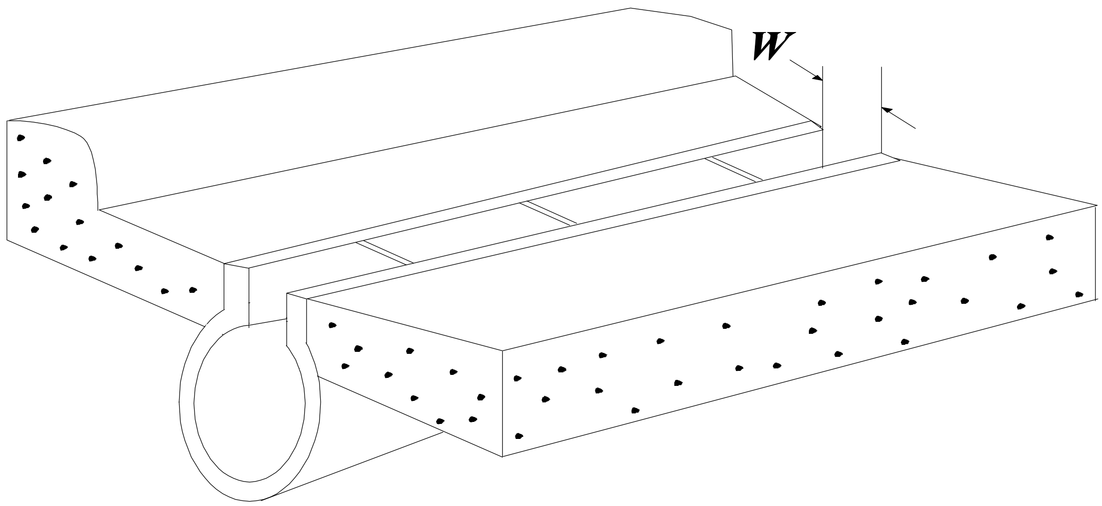

Figure 6. Perspective view of slotted drain inlet.[1]

Grate Inlet

Grate inlets on grade, as shown in figure 4, will intercept all of the gutter flow passing over the grate, or the frontal flow, if the grate is sufficiently long and the gutter flow velocity is low. Only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. A part of the flow along the side of the grate will be intercepted, depending on the cross slope of the pavement, the length of the grate, and flow velocity.

The ratio of frontal flow to total gutter flow, E_o, for a uniform cross slope is calculated by equation (13):

$$E_o = \frac{Q_w}{Q} = 1 - \left[1 - \frac{W}{T}\right]^{\frac{8}{3}} \tag{13}$$

Where:

Q	=	Total gutter flow, m^3/s.
Q_w	=	Flow in width W, m^3/s.
W	=	Width of depressed gutter or grate, m.
T	=	Total spread of water in the gutter, m.

The ratio of side flow, Q_s, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{14}$$

The ratio of frontal flow intercepted to total frontal flow, R_f, or frontal flow efficiency, is calculated by equation (15):

$$R_f = 1 - 0.295 \times (V - V_o) \tag{15}$$

Where:

V	=	Velocity of flow in the gutter, m/s.
V_o	=	Gutter velocity where splash over first occurs, m/s.

The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is calculated by equation (16):

$$R_s = \frac{1}{\left[1 + \frac{0.083 \times V^{1.8}}{S_x \times L^{2.3}}\right]} \tag{16}$$

Where:

L	=	Length of the grate, m.
S_x	=	Cross slope of pavement, m/m.
V	=	Velocity of flow in the gutter, m/s.

The efficiency, E, of a grate is calculated by equation (17):

$$E = R_f \times E_o + R_s \times (1 - E_o) \tag{17}$$

The first term on the right side of equation (17) is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity of a grate inlet, on grade, is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = E \times Q = Q \times \left[R_f \times E_o + R_s \times (1 - E_o)\right] \tag{18}$$

For the analysis case, equations (13) through (18) need only be applied one time. Since the inlet dimensions are known, the interception capacity can be determined directly. Flows in excess of the grate capacity are stored for later recall as specified by the user in the **INL** command.

In order to design a grate inlet length to accept a design flow, on grade, HYDRA iterates through equations (13) through (18), increasing the grate length until the flow calculated by equation (18) is within 1/2 of 1 percent of the design flow, or the design length reaches 6.10 m. The tolerance is employed because, in cases where the width of flow is greater than the width of the inlet grate, then the percentage of flow captured by the inlet can never be 100 percent. This occurs, computationally, because the R_s term in equation (18), which represents the amount of side flow captured, will always be less than 1.0. Even though it can get very small, it will always reduce the efficiency of the grate, making 100 percent capture impossible. Thus an infinite grate length would be required. By assuming that 99.5 percent capture is sufficient, reasonable grate lengths can be achieved. If the width of flow is less than or equal to the grate width, 100 percent capture is achieved with no unrealistic computational problems occurring.

The second condition described above, which will not allow a grate design of greater than 6.10 m, is included as a practical limitation. The design equations are empirical in nature, based on experiments providing design guidance for grates up to 1.20 m in length. While there is no reason to doubt the validity of the design equations for lengths greater than 1.20 m, conditions requiring excessive grate lengths should be examined.

A grate inlet in a sag (sump) location operates as a weir, to depths dependent on the bar configuration and size of the grate, and as an orifice at greater depths. Grates of larger dimension and grates with more open area, i.e., with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than smaller grates or grates with less open area. HYDRA calculates the maximum depth at which weir flow will take place:

$$weir\ depth_{\max} = 0.08 + (0.02 \times P) \tag{19}$$

Where:

P = Perimeter of the grate, disregarding bars and the side against the curb, m.

Also calculated is the minimum depth at which orifice flow will occur:

$$orifice\ depth_{\min} = 0.265 + (0.013 \times A) \tag{20}$$

Where:

A = Clear opening area of the grate, m^2.

The inlet depth is then compared to the values computed above to determine the governing flow regime. Once this is established, the correct equation can be applied to calculate the capacity of the inlet. For cases where the flow is transitional, i.e., neither weir flow nor orifice flow clearly dominates, the minimum flow computed from the orifice and weir equations is selected.

In analyzing a grate inlet of given dimension, the capacity, assuming the inlet is operating as a weir, is computed as:

$$Q_i = C_w \times P \times d^{\frac{3}{2}} \tag{21}$$

Where:

C_w	=	The weir coefficient (equal to 1.66).
d	=	Depth of water, m.

Since the grate dimensions are known for the analysis condition, the perimeter is computed as 2 × grate width + grate length.

The capacity of a grate inlet operating as an orifice is computed as:

$$Q_i = C_o \times A \times \sqrt{2 \times g \times d} \tag{22}$$

Where:

C_o	=	Orifice coefficient equal to 0.67.
g	=	Acceleration due to gravity equal to 9.81 m/s^2.
d	=	Depth of water, m.

The depth, **d**, must be estimated to use equations (21) or (22). HYDRA assumes that **d** is approximated by the cross slope, $\mathbf{S_x}$, multiplied by the spread, **T**.

The clear opening area of the grate is calculated as the product of the grate width, the grate length, and the opening ratio. The clear opening area, as well as the perimeter, can be overridden by entering these values in the **INL** command. In this manner, the potential effects of clogging can be studied. The opening ratio is the ratio of open area to the area of the bars projected to a horizontal plane. As an example, a grate inlet with 30-degree tilt bars would have a higher opening ratio than the same grate with 45-degree tilt bars. In fact, a 45-degree tilt bar grate is not recommended for sump situations because the opening ratio is so small. While it is clear some flow does enter such a grate, technically speaking the opening ratio is zero. Thus, if a 45-degree tilt bar subtype is selected in a sump situation, the program uses the opening ratio for a 30-degree tilt bar. The opening ratios used in HYDRA are taken from HEC-12 and are shown in table 1.[(1)]

Table 1. Opening ratios for various grate inlets.

Grate	**Opening Ratio**
P-1-7/8-4	0.80
P-1-7/8	0.90
P-1-1/8	0.60
Reticuline	0.80
Curved Vane	0.35
30-degree Tilt-bar	0.34
45-degree Tilt-bar	0.00

To design the size of a grate inlet in a sump situation, HYDRA solves equation (21) for the required perimeter and equation (22) for the clear opening area. The type of flow regime is then computed, again by comparing the flow depth to equations (19) and (20), and either the perimeter or the area is solved for the required grate length.

Curb Inlet

Curb-opening inlets, as shown in figure 5, are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet, on grade, necessary to intercept 100 percent of flow in the gutter is computed by:

$$L_T = K \times Q^{\frac{5}{12}} \times S^{\frac{3}{10}} \left[\frac{1}{n \times S_e}\right]^{\frac{3}{5}} \qquad (23)$$

Where:

K	=	Coefficient equal to 0.81.
Q	=	Gutter flow, m^3.
S	=	Longitudinal slope (vertical to horizontal), m/m.
n	=	Roughness coefficient.
S_e	=	Equivalent cross slope (vertical to horizontal), m/m.

The equivalent cross slope, S_e, is computed by the following equation:

$$S_e = S_x + S'_w \times E_o \tag{24}$$

Where:

S'_w	=	Cross slope of the gutter measured from the cross slope of the pavement, S_x, computed as a/w or S_w - S_x.
a	=	Inlet depression, m.
w	=	Width of depression, m.
E_o	=	Ratio of flow in the depressed section to total gutter flow (equation (13)).

In the analysis mode, equation (23) is solved for **Q** to determine the amount of flow the inlet of specified length can accept. Flows exceeding the inlet capacity are stored for later recall as specified by the user in the INL command.

The capacity of a curb-opening inlet in a sag or sump depends on water depth at the curb, the curb-opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The weir location for a depressed curb-opening inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb opening. The weir location for a curb-opening inlet that is not depressed is at the lip of the curb opening, and its length is equal to that of the inlet. Limited experiments and extrapolation of the results of tests on depressed inlets indicate that the weir coefficient for curb-opening inlets without depression is approximately equal to that for a depressed curb-opening inlet.

A weir equation is applicable to depths at the curb less than or equal to the height of the opening plus the depth of depression. HYDRA calculates the interception capacity of a depressed curb opening inlet in a sag or sump as:

$$Q_i = C_w \times (L + 1.8 \times W) \times d^{\frac{3}{2}} \tag{25}$$

Where:

C_w	=	Weir coefficient (equal to 1.27).
L	=	Length of curb opening, m.
W	=	Lateral width of depression, m.
d	=	Depth at curb measured from the normal cross slope, m.

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the height of the curb opening. The orifice equation for curb-opening inlets is used for d greater than h and is calculated by:

$$Q_i = C_o \times h \times L \times \sqrt{2 \times g \times d_o} \tag{26}$$

Where:

C_o	=	Orifice coefficient (equal to 0.67).
h	=	Height of curb-opening inlet, m.
d_o	=	The effective head on the center of the orifice throat, m.
L	=	Curb-opening inlet length, m.

In designing the length of curb-opening inlet for a given flow, HYDRA solves one of the above equations depending on the flow regime as defined by the depth. The necessary length is calculated from equation (25) or (26) as is appropriate.

Slotted Inlets

Slotted inlets, as shown in figure 6, are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Thus, the calculations for designing or analyzing a slotted inlet on grade are the same as for a curb inlet (equation (23)).

Slotted inlets in sag locations perform as weirs to depths of about 61 mm, depending on the slot width and length. For these, the following condition applies:

$$Q_i = C_w \times (L + 1.8 \times W_s) \times d^{\frac{3}{2}} \tag{27}$$

Slotted inlets in sag locations perform as orifices at depths greater than about 122 mm. The interception capacity of a slotted inlet operating as an orifice is computed by equation (28):

$$Q_i = C_o \times L \times W_s \times \sqrt{2 \times g \times d} \tag{28}$$

Where:

C_o	=	Orifice coefficient (equal to 0.8).
W_s	=	Width of slot, m.
L	=	Length of slot, m.
d	=	Depth of water at slot, m.

g = Acceleration due to gravity equal to 9.81 m/s^2.

For transitional flow depths (between 61 mm and 122 mm), equation (28) is used with one modification: the coefficient of 0.8 is set to 1.0.

User-Defined Inlets

The user has the option of analyzing the performance of an inlet of a type other than the three described above. This is accomplished through the use of the **INL** command immediately followed by the **EFF** command. The latter command defines the flow-efficiency relationship of a nonstandard inlet type. This curve is consulted to determine what percentage of the flows generated by HYDRA are accepted into the inlet.

INLET BYPASS

When gutter flow arriving at the inlet exceeds the inlet capacity, there is an excess in flow, or bypass flow. This bypass flow can be captured at another point downstream in the system for both the hydrographic and peak flow simulations. The combination of the **store** parameter (**INL** command) and the **GET** command accomplish this application of storing the bypass flow to a register and retrieving the bypass flow, respectively. The **GUT** command must immediately follow the **GET** command in a bypass flow application. However, in the case of the peak flow analysis, the system flow is not affected by a bypass flow simulation (i.e., even though, theoretically, the inlet does not have the capacity to convey all of the gutter flow through the inlet and into the system transporter, the model assumes all of the flow does enter the system). In the context of the peak flow analysis, bypass flow is only an instrument to evaluate inlet efficiency.

FLOW STORAGE

The user has the option of including the effects of storage on surface flows as well as on system flows with the use of the **PON** and **RES** commands, respectively. Both commands are operational only during hydrographic runs.

The **PON** command allows for surface ponding of flows from the **UHY**, **GET**, or **GUT** commands. The user has two options; to determine the:

1. Necessary pond capacity given a maximum outflow.
2. Maximum outflow given a pond capacity.

For option number 1, hydrographic flows exceeding the specified outflow are stored. These flows accumulate over the length of the hydrograph. The product of the excess flow and the time step yields the volume for each time step. The summation is equal to the required pond capacity. This approach assumes that all the excess flow volume can be stored.

For option 2, the hydrograph is truncated with the volume of the truncated portion equal to the given pond capacity. The flow of the hydrograph at this truncated level is the maximum return rate. The user is also informed of the duration at which the pond had any effect on the hydrograph.

The **RES** command allows the user to study the effects of storage on system flows, i.e., flows in a channel or pipe. The user has three options, described below:

1. Design the required capacity given a maximum outflow.
2. Determine the adequacy of a reservoir size given capacity and maximum outflow.
3. Perform a routing through the reservoir using the storage indication routing method.

For option 1, when hydrographic flows start to exceed the maximum outflow, the reservoir starts to fill. The summation of flows exceeding the maximum outflow minus the maximum outflow determines the required reservoir capacity.

If the user chooses a reservoir capacity (option 2) and it is not large enough to accept the entire hydrograph, flows in excess of the capacity will be bypassed. The user will be informed that this has occurred via a message in the output.

Option 3 allows the user to perform a routing through the reservoir using the storage indication (modified Puls) routing method.[9] The stage-storage and stage-discharge curves for the reservoir are entered in the **SST** and **SDI** commands, respectively. HYDRA then solves equation (29) for the inflow and outflow at the end of each time step:

$$\Delta s = \left[\frac{I_i + I_j}{2} - \frac{O_i + O_j}{2} \right] \times \Delta t \tag{29}$$

Where:

Δs	=	The change in storage, m^3.
I_i	=	The inflow at the beginning of the time step, m^3/s.
I_j	=	The inflow at the end of the time step, m^3/s.
O_i	=	The outflow at the beginning of the time step, m^3/s.
O_j	=	The outflow at the end of the time step, m^3/s.
Δt	=	Time step, s.

Equation (29) assumes linearity of flow from the beginning to the end of the time step. This is a good assumption for small time increments. As the time increment increases, the assumption of linearity becomes less valid, resulting in mathematical errors in the routing procedure.

COST ESTIMATING

HYDRA, through the use of criteria established in the following commands, performs a cost estimate to design pipe in place:

(1) **CST** - ditch geometry and unit prices for material and haul.
(2) **EXC** - ditch excavation costs.
(3) **PCO** - pipe costs.
(4) **TSL** - establishes trench side slopes.
(5) **ECF** - extra excavation costs.
(6) **PCF** - extra pipe costs.
(7) **LPC** - summary table of unit costs and materials.

In order for HYDRA to calculate costs for pipe in place, the first four of the above commands must be utilized; the remaining three are optional. HYDRA performs numerous calculations to arrive at the final cost estimates. These estimates are calculated by first determining the amount of a given material and then multiplying that value by its unit cost. This process is carried out on a link by link basis. As a general representation, cost can be said to be equal to the following:

$$cost = \sum \left(X_i \times U_i\right) + lcost \tag{30}$$

Where:

cost	=	The cost associated with the sanitary sewer or storm drain project, dollars.
X_i	=	The quantity of excavation, backfill material, pipe zone material, bedding material, and pipe.
U_i	=	The cost of excavation, backfill material, pipe zone material, bedding material, and pipe, dollars per item or quantity.
lcost	=	Lump sum costs (e.g. inlets, pumps), dollars.

The “lump sum costs” term includes monies that are simply added on directly. Included in this category is the **lcost** term in the **PIP**, **ELP**, and **BOX** commands as well as the **cost** parameter specified in the **PUM** command.

HYDRAULIC GRADELINE METHODOLOGY

The user has the option, through the use of **HGL** command, of initiating the calculation of the hydraulic gradeline through the system under investigation. Using information supplied in the **PNC**, **PIP**, **BOX**, and **ELP** commands concerning pipe-node connectivity and characteristics, the calculations proceed from the system outfall upstream to each of the terminal nodes. Calculation of the hydraulic gradeline includes the determination of major and minor losses within the system. Major losses result from friction losses within the pipe. Minor losses include those losses attributed to bends in pipes, manhole losses, expansion and contraction losses, and losses at appurtenances such as valves and meters.

The detailed methodology employed in calculating the hydraulic gradeline through the system begins at the system outfall with the tailwater elevation. This value can be determined in one of two ways: (1) the flow depth in the outfall link as calculated by HYDRA, or (2) input by the user via the **TWE** command. It is important to note that the program detects the outfall, and hence where to begin the hydraulic gradeline calculations, by reading a node width (diameter of the manhole) of **0** or node type of **2** in the relevant **PNC** command. If a node width of **0** or a node type of **2** is not entered for the outfall point, the hydraulic gradeline calculations will not take place.

Once the tailwater elevation is established, HYDRA checks to see if the value is greater than or equal to the crown elevation of the downstream end of the outfall link. If the above condition is true, HYDRA assumes that the pipe is surcharged and calculates, using Manning's equation, the friction slope necessary to achieve the calculated flow. This slope is then multiplied by the length of the pipe to estimate the major friction losses. Any additional pipe losses (either bend losses, as detailed in the **BEN** command, or user-supplied losses, as input in the **LOS** command) are added. This elevation is compared to the summation of the flow depth, as calculated by HYDRA, plus the upstream invert elevation. The greater of the two values is then assumed to be the hydraulic gradeline at the upstream end of the pipe. If the outfall link is not flowing full, then the potential hydraulic gradeline elevation at the upstream end is calculated as the flow depth plus the upstream invert elevation.

Minor losses occurring as flow passes through a manhole are determined for the particular connection. This value is then added to the hydraulic gradeline of the downstream pipe to arrive at the hydraulic gradeline which is experienced just inside the upstream pipe, and is compared with the crown elevation of the upstream pipe. At this point, a new "tailwater elevation" has been calculated for the next upstream pipe. This process is repeated for each pipe on the system.

Minor losses are calculated for both manholes and pipe junctions. A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of a manhole structure. For adjoining pipes to be considered a pipe junction, the node type must be specified as **1** on the **PNC** command and only two inflow pipes (a lateral and a trunk) may enter the junction (for more than two pipes, the manhole loss equations are employed). The minor loss equation for a pipe junction is a form of the momentum equation:

$$H_j = \frac{Q_o \times V_o - Q_i \times V_i - Q_l \times V_l \times \cos\,\theta}{0.5 \times g \times (A_o + A_i)} + h_i - h_o \tag{31}$$

Where:

H_j	=	Junction head loss, m.
Q_o, Q_i, Q_l	=	Outlet, inlet, and lateral flows, respectively, m^3/s.
V_o, V_i, V_l	=	Outlet, inlet, and lateral velocities, respectively, m/s.
h_o, h_i	=	Outlet and inlet velocity heads, m.
A_o, A_i	=	Outlet and inlet cross-sectional areas, m^2.
θ	=	Angle of lateral with respect to centerline of outlet pipe, degrees.
g	=	Gravitational acceleration, 9.81 m/s^2.

MINOR LOSS: HYDRAULIC GRADELINE ANALYSIS

The hydraulic gradeline analysis through a manhole focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe.[12] A water depth in the manhole is estimated using an iterative procedure. Subsequently, the energy losses for each inflow pipe may be computed using the manhole depth and discharge.

The hydraulic gradeline methodology starts with equation (32) which describes the energy loss for an inflow pipe:

$$\Delta E = K_I \frac{V_o^2}{2g} \tag{32}$$

The methodology is completed by using the determining factors yielded from a dimensional analysis to predict K_I for a given physical configuration and hydraulic loading:

$$K_I = (C_1 \ C_2 \ C_3 + C_{4_i})\omega \tag{33}$$

Where:

K_I = Composite energy loss coefficient for an inflow pipe.
C_1 = Coefficient related to relative manhole size.
C_2 = Coefficient related to water depth in the manhole.
C_3 = Coefficient related to lateral flow, lateral angle, and plunging flow.
C_{4_i} = Coefficient related to relative pipe diameters.
ω = Correction factor for benching.

Equations (32) and (33) are combined to yield the following equation:

$$\Delta E_I = (C_1 \ C_2 \ C_3 + C_{4_i})\omega \ \frac{V_o^2}{2g} \tag{34}$$

Determining Factors

Several determining factors affect the computation of the energy loss coefficient in the HGL methodology. They are the manhole size relative to the outlet pipe diameter, depth in the manhole, amount of inflow, inflow angle, plunge height, relative pipe diameter, and floor configuration. Empirical equations for each of these determining factors are developed from analysis of data collected by Chang.[12]

Relative manhole size

The role of relative manhole diameter (manhole diameter/outlet pipe diameter, b/D_o) is evaluated using data having a single inflow pipe at the same invert as the outflow pipe representing straight-through flow. The energy loss coefficient, in this case, increases with relative manhole size. The larger the manhole is relative to the outlet pipe, the greater the space and time are for the flow to expand and dissipate the velocity head. Similarly, the greater the expansion into the manhole, the greater the energy losses in contracting to leave through the outlet pipe. According to Marsalek, the energy loss coefficient is unaffected by changes in relative manhole diameter within the range of $2 \le b/D_o \le 6$. Sangster's study showed that the energy loss coefficient is more affected in the range of $b/D_o < 3$.[11,13] The proposed formulation for this methodology is that for b/D_o values up to 4.0, the coefficient related to manhole size, C_1, is calculated with the following equation:

$$C_1 = \frac{0.9\left[\dfrac{b}{D_o}\right]}{\left[6 + \dfrac{b}{D_o}\right]} \tag{35}$$

Where:

b	=	Manhole diameter, m.
D_o	=	Outflow pipe diameter, m.

The data from Sangster and Marsalek suggest that the relationship between the coefficient C_1 and the relative manhole diameter, b/D_o is nearly linear up to the point where b/D_o is approximately 4.0. Beyond this value, the relative manhole diameter has no effect on the head loss coefficient, C_1. Once the manhole diameter is four times the outlet pipe diameter, or larger, the manhole is "large" and the coefficient C_1 is assumed to be a constant equal to 0.36.

Water depth in the manhole

The coefficient, C_2, related to manhole water depth, increases rapidly with relative water depth, d_{mH}/D_o, up to 2. The rate of the increase slows when d_{mH}/D_o reaches approximately 3. This type of curve can be expressed as a third order polynomial. Equation (36) applies for $d_{mH}/D_o \leq 3$. When d_{mH}/D_o is greater than 3, C_2 is equal to 0.82. The following equation was found to fit the data reasonably well:

$$C_2 = 0.24 \left(\frac{d_{mH}}{D_o} \right)^2 - 0.05 \left(\frac{d_{mH}}{D_o} \right)^3 \qquad (36)$$

Where:

d_{mH} = Depth in the manhole relative to the outlet pipe invert, m.

Multiple inflows

The coefficient related to multiple inflows, C_3, is the most complex term in the composite energy loss coefficient equation. The effect of lateral flows on the energy loss were studied with respect to three parameters: flow rate, connecting angle of the inflow pipe, and elevation of the inflow pipe. To select the form of the equation, the data were first plotted in groups according to the above three parameters to observe the variations in energy losses. The data are quite scattered because of air entrainment and turbulence. This was particularly true for plunging flows discharging into shallow water depths in the manhole. Based on the analysis, the following equation can be used to calculate the coefficient C_3.

$$C_3 = \text{Term 1} + \text{Term 2} + \text{Term 3} + \text{Term 4} + \text{Term 5} \qquad (37)$$

Where:

Term 1 = 1

$$\text{Term 2} = \sum_{i=1}^{4} \left(\frac{Q_i}{Q_0} \right)^{0.75} \left[1 + 2\left(\frac{Z_i}{D_0} - \frac{d_{mH}}{D_0} \right)^{0.3} \left(\frac{Z_i}{D_0} \right)^{0.3} \right]$$

$$\text{Term 3} = 4 \sum_{i=1}^{3} \frac{\left(Cos\theta_i \right) \left(HMC_i \right)}{\left(\frac{d_{mH}}{D_0} \right)^{0.3}}$$

$$\text{Term 4} = 0.8 \left| \frac{Z_A}{D_0} - \frac{Z_B}{D_0} \right|$$

$$\text{Term 5} = \left| \left(\frac{Q_A}{Q_0} \right)^{0.75} Sin\theta_A + \left(\frac{Q_B}{Q_0} \right)^{0.75} Sin\theta_B \right|$$

$$HMC_i = \left[0.85 - \left(\frac{Z_i}{D_o} \right) \left(\frac{Q_i}{Q_o} \right)^{0.75} \right] \tag{38}$$

and:

Q_0	=	Total discharge in the outlet pipe, m^3/s.
Q_1, Q_2, Q_3	=	Pipe discharge in inflow pipes 1, 2 ,and 3, m^3/s.
Q_4	=	Discharge into manhole from the inlet, m^3/s.
Z_1, Z_2, Z_3	=	Invert elevation of inflow pipes 1, 2, and 3 relative to the outlet pipe invert, m.
Z_4	=	Elevation of the inlet relative to the outlet pipe invert, m.
D_0	=	Outlet pipe diameter, m.
b	=	Manhole diameter, m.
d_{mH}	=	Depth in the manhole relative to the outlet pipe invert, m.
$\theta_1, \theta_2, \theta_3$	=	Angle between the outlet main and inflow pipes 1, 2, and 3, degrees.
HMC_i	=	Horizontal momentum check for pipe i.
Q_A, Q_B	=	Pipe discharges for the pair of inflow pipes that produce the largest value for term 4, m^3/s.
Z_A, Z_B	=	Invert elevation, relative to outlet pipe invert, for the inflow pipes that produce the largest value for term 4, m.

The coefficient, C_3, takes into consideration losses due to inlet flow plunging by incrementing the number of inflows to four. The fourth inflow pipe is synthetic and accounts for inlet plunging. A corresponding angle is set to zero. Conceptually, as angles deviate from 180 degrees (straight-line flow) to 0 or 360 degrees, the associated loss increases because the skewed inflow prevents the main flow from smoothly transitioning to the outlet pipe. **All angles are represented between 0 and 360 degrees for this equation and are measured clockwise from the outlet pipe** as illustrated in Figure 7.

A pipe has plunging flow if the critical flow depth elevation ($y_c + Z_i$) in the pipe is higher than the manhole depth elevation ($d_{mH} + Z_o$). For a simple two-pipe system with no plunging flow, C_3 is assumed to be equal to 1.0.

The second term in equation (37) captures the energy losses from plunging inflows. This term reflects the fact that flows plunging from greater heights result in greater turbulence and, therefore, higher energy losses. As shown by the summation in the second term, the computation is valid for one to three inflow pipes and plunging flow from the inlet.

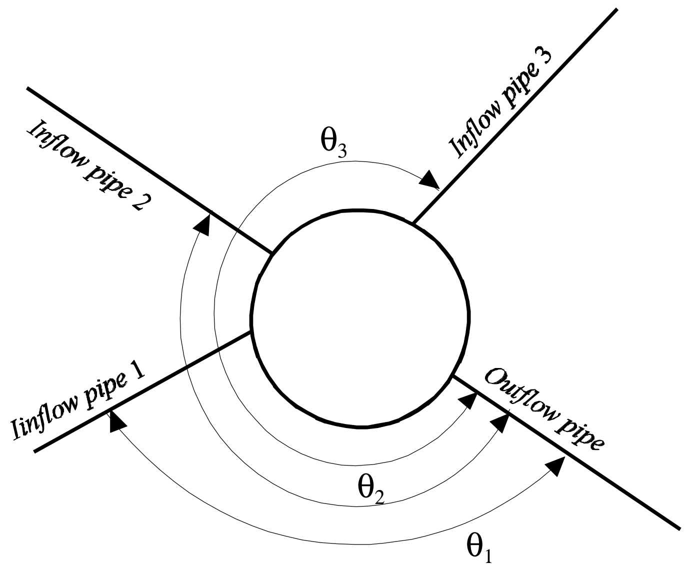

Figure 7. Convention of angle of deflection.

The third term reflects the effects the angle (with respect to the outflow pipe) has on energy losses. For inflows at a given invert (Z_i) and relative discharge (Q_i / Q_0), the cosine function provides for a higher energy loss coefficient when the inflow opposes ($\theta \leq 90$ degrees or $0 \geq 270$ degrees) the main inflow. If the horizontal momentum check (HMC_i) is less than 0, the flow is falling from a height such that the horizontal momentum is assumed to be negligible and term 3 is set to 0. Since surface flow is free-falling and has no horizontal momentum, it is not included in term 3 calculations.

The invert elevation, manhole depth, and relative inflow are also included in the third term to capture extremely complex interactions. For a 135-degree skewed plunging flow, for example, the velocity component of the flow in the main flow direction increases which results in lower energy losses because the momentum is predominantly in the direction of the outflow pipe. But, a higher flow velocity creates higher turbulence, thereby increasing the energy loss. If the skewed pipe is raised further so that the flow hits the opposite wall of the manhole during the fall, a different situation develops. The plunging flow dives along the manhole wall and moves in the opposite direction as the flow reaches the manhole bottom. The velocity component of the flow at the manhole bottom is then in the opposite direction it entered reversing the energy loss effect. In cases where water depth in the manhole is significant, the velocity of the diving flow will be reduced.

When more than one inflow pipe exists, HYDRA computes the fourth term for all combinations of inflow pipes that have an HMC_i greater than zero. HYDRA then uses the pipes that produce the highest fourth term value to calculate the fourth and fifth terms. If there are at least two inflow pipes with positive HMC_i values, the third, fourth, and fifth terms are calculated; otherwise they are assumed to be zero.

Inspection of the fourth and fifth terms of equation (37) reveals that if two inflow pipes enter a junction manhole 180 degrees apart from one another, with equal discharges, and at the same invert elevation, they both reduce to zero. This means that opposing inflow pipes tend to neutralize, to a degree, the turbulence each would cause individually. For cases where discharge, angle, and/or elevation are different, additional energy losses occur.

Application limits for equation (37) with plunging flow ranges from 1.0 (no lateral flow) to 10.0. The predicted C_3 coefficient increases dramatically as the height of the plunging inflow increases.(12) In reality, the estimate of the C_3 coefficient from the empirical data does increase at higher plunge heights, but the data are very scattered. In other words, the empirical data does not support the dramatic upward trend that equation (37) predicts at higher plunge heights. Therefore, the chosen upper limit of 10.0 is a realistic ceiling for plunging flow.(14)

Relative pipe diameters

Although no experiments were performed with different pipe diameters in this study, a correction for such a case is required in the hydraulic gradeline analysis. Equation (39) was theoretically derived based on conservation of momentum and is proposed for this purpose. Since the C_4 term represents an exit loss from each inflow pipe, it is calculated for each pipe that does not have plunging flow. C_{4i} is assumed to be zero for pipes with plunging flow ($Z_i + yc_i > Z_o + d_{mH}$). Each loss is unique to each inflow pipe and does not affect any other inflow pipes.

$$C_{4i} = 1 + \left[\left(\frac{Q_i}{Q_o} + 2 \, \frac{A_i}{A_o} \cos \theta_i \right) \frac{V_i^2}{V_o^2} \right] \qquad (39)$$

Where:

A_i,A_o = Cross-sectional area of inflow and outflow pipes, m^2.
θ_i = Angle between outflow pipe and inflow pipe i, degrees.

If θ_i for any pipe is less than 90 degrees or greater than 270 degrees, ($\cos \theta_i$) is replaced with 0 in equation (39). This sets the maximum exit loss to be the incoming velocity head. The derivation of equation (39) followed theoretical derivation presented by Sangster, but Sangster's expression, which was used in earlier versions of HYDRA, was limited to a two-pipe system with pipes flowing full at 180-degree angles.

Equation (39) is consistent with Sangster's findings and is a more generalized form of the equation. A higher loss coefficient for pipe diameter reflects increasingly constricted flow in the inlet pipe for a given outlet pipe diameter and velocity head. Energy lost as a result of differing pipe diameters is significant only in pressure flow situations when the depth in the manhole is greater than the outlet pipe diameter. The upper limit of C_4 used in HYDRA is 9.0.

Floor configuration

Manhole benching affects head loss because the bending channels guide the flow smoothly into the outlet pipe reducing disturbance in the flow. The reduction in the head loss is dictated by the type and extent of benching. The reduction of head loss is greater when all incoming flow is smoothly reflected by the benching facility.

Marsalek tested three types of benchings, exhibited in figure 8, that are commonly used in drainage practice. One type is a half benching for which the lower half of the pipe extends through the junction and horizontal benches are extended from the semi-circular channel to the junction wall. In the plan view, the channel axis follows a 90-degree segment of a circle with a radius equal to one half of the manhole diameter. The second type is a full benching which is an improved variation on the half benching obtained by extending the mold side wall to the pipe crown elevation. The third type is an improved variation of the full benching which adds smooth transition sections to the inflow and outflow pipes. The inflow was gradually enlarged 30 percent in diameter and then smoothly contracted into the outflow pipe.

His test data showed that the half and the full benchings reduced head losses by 5 percent and 25 percent, respectively, for pressure flows. For free-surface flows, the reductions were 85 percent and 93 percent, respectively. The improved full benching experienced a reduction rate of 60 percent for the pressurized flows and nearly 100 percent for the open-channel flows. It is no surprise that the reduction rates were different for these flows. The water depths were shallow for the free-surface flow, and thus, all or most of the flows were confined within and guided through the molded channel smoothly. For the pressure flow, the water in the manhole was deep, and the incoming flow was not totally confined and allowed to expand upward, causing agitation. Therefore, the energy-loss reductions were not as significant. The final selection of the type of benching depends on economics. There will be a substantial reduction in the head loss for the improved benching, however, a question of the additional cost required should be addressed in making the final decision. For half benching, or even for full benching, the improvement as related to the reduction in the head losses of 5 percent or 25 percent may not be sufficient to override the extra cost required to mold the bench.

In practice, the correction factors obtained from Marsalek's data and shown in table 2 may be used. The correction factors, ω, should be multiplied by the head-loss coefficient for a manhole with a flat floor. For conditions that are between clear pressure flow ($d_{mH} / D_o > 3.2$) and clear free-surface flow ($d_{mH} / D_o < 1.0$), a linear interpolation is an appropriate approximation.

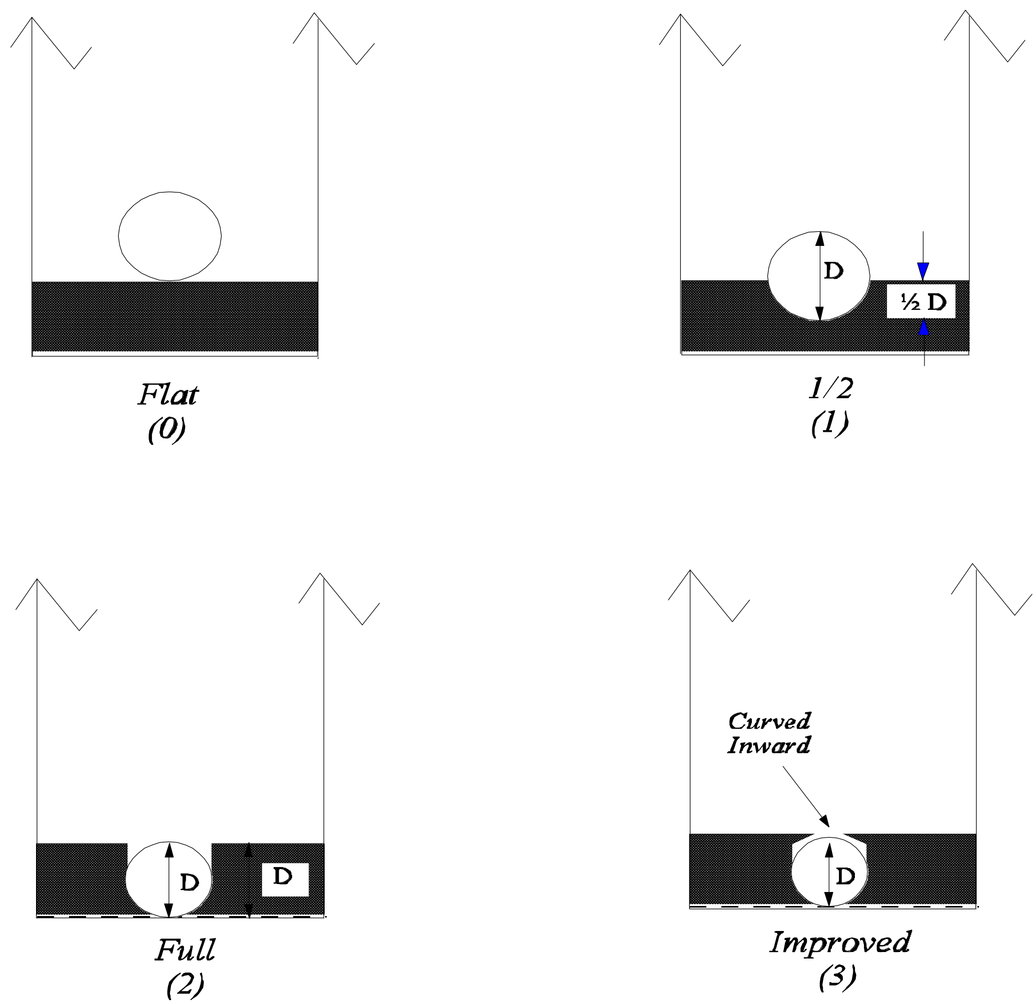

Figure 8. Schematic representation of benching types.

Table 2. Correction factors, ω, for benching.

	Bench Submerged*	Bench Unsubmerged**
Flat Floor	1.0	1.0
Benched one-half pipe diameter high	0.95	0.15
Benched one pipe diameter high	0.75	0.07
Improved	0.40	0.02

*   pressure flow, $d_{mH} / D_0 > 3.2$

**   free-surface flow, $d_{mH} / D_0 < 1.0$

APPLYING THE METHODOLOGY

The methodology developed can be applied by determining the estimated energy loss through a junction manhole given a set of physical and hydraulic parameters. Computation of the energy loss allows for the determination of the hydraulic gradeline upstream of the manhole to be analyzed. After the depth in the manhole is determined, the HGL for the inflow pipes are computed.

The user should note that this methodology applies to subcritical flow in pipes. When pipes are flowing supercritical, the HGL computations can be excessively high. This occurrence is due to the iterative solution of the depth in the manhole and the assumption that the downstream manhole depth is the hydraulic control. For pipes that are supercritical, the HGL computations assume critical depth in the pipe and the profile computations continue upstream.

Solving for Manhole Depth

To determine the depth in the manhole, an iterative procedure must be applied in computing the energy loss because the manhole depth is a function of C_2 and C_3, which, in turn, are functions of d_{mH} (which is unknown).

$$d_{mH} = HGL_o + \frac{V_o^2}{2g} + C_1 C_2 C_3 \; \omega \; \frac{V_o^2}{2g} - Z_o \qquad (40)$$

Where:

HGL_o	=	Hydraulic gradeline at the upstream end of the outlet pipe, m.
Z_o	=	Invert elevation of outlet pipe at upstream end, m.

As an initial estimate of d_{mH}, the following equation is used:

$$d_{mH} = HGL_o + C_1 \omega \frac{V_o^2}{2g} - Z_o \qquad (41)$$

Use of C_1 as that initial estimate is reasonable since it is independent of d_{mH}; calculation of C_2 and C_3 proceed according to the appropriate equations. Equation (40) is applied to determine a manhole depth. The computed d_{mH} value is compared to the prior estimate, and the procedure is repeated until these two values converge. If the junction manhole configuration has only one inflow pipe that is not plunging, d_{mH} may be directly calculated.

Computation of the HGL for inflow pipes

Once the manhole depth is calculated, the computed junction loss, ΔE, is applied so that the hydraulic gradeline in the inflow pipe(s) may be computed. The hydraulic gradeline at the downstream end of the inflow pipe is assumed to be the greatest value from equations (42), (43), and (44).

$$HGL_i = d_{mH} + Z_o \tag{42}$$

$$HGL_i = Z_i + d_i \tag{43}$$

$$HGL_i = HGL_o + \Delta E_i \tag{44}$$

Where:

HGL_I = Hydraulic gradeline at the outlet of inflow pipe, i, m.
Z_i = Invert elevation at the outlet of inflow pipe, i, m.
d_i = Normal depth (subcritical pipe) or critical depth (supercritical pipe) of pipe, i, m.

PRESSURE FLOW SIMULATION

Using HYDRA, a user may simulate performance of a system under pressurized (surcharged) flow conditions. This dynamic analysis methodology allows consideration of the effects of pipe storage and pressure surcharges in the system response to a hydrograph. Such a dynamic analysis requires additional information from the user, but for some drainage systems, may demonstrate that a system will perform adequately when more conservative steady-state approaches will not. The pressure flow computations, like the HGL computations, use a link-node description to represent the physical system.(4) Hence, the **PIP**, **ELP**, **BOX**, and **PNC** commands, once again, provide information concerning pipe-node connectivity and characteristics. The system's outfall assumes a tailwater elevation, either specified by the user on the **TWE** command or the default value, the downstream invert of the outlet pipe.

The pressure flow simulation can be initiated for systems modeled with hydrographic flow, using **UHY**, or steady-state flow, employing **STO**. Hydrographic flow is accepted into the system at a junction by way of an inlet using the **INL** command. However, the Rational method flow from the **STO** command can be introduced into the system at a junction with or without the use of the **INL** command.

Associated with the pressure flow computations are five additional commands: the pressure flow simulation command, **PFS**; the initial depth command, **IDY**; the initial flow and velocity command, **IQV**; the pipe flow print command, **PFP**; and the junction head print command, **PHJ**. The **PFS** command initiates the computations and establishes the flow control parameters. The optional initial condition commands, **IDY** and **IQV**, introduce the depths and flows and velocities, respectively, at the start of simulation. The print commands, **PFP** and **PHJ**, provide the user detailed output of specified pipe and junction results, respectively.

The pressure flow module recognizes the pipe-node connectivity of a system such that pipes transmit flow from one node to another node. These pipes have the following properties: roughness, length, cross-sectional area, hydraulic radius, and surface width. The last three properties are dependent on the instantaneous depth of flow.(4) The primary dependent variable in the pipes is the discharge, **Q**. It is assumed that **Q** is constant throughout the entire link during a computational time step, while velocity and the cross-sectional area of flow may vary within the link.

Nodes are the storage elements of the system and correspond to manholes or conduit junctions in the physical system. The variables associated with a node are volume, head, and surface area. The primary dependent variable is head, **H**, assumed to be constant within each node during a computational time step. Inflows, such as inlet hydrographs, and outflows take place at the nodes of the conveyance system. The volume of water in a node is the sum of the water volume in the half-pipe lengths connected to that node. The change in nodal volume during a given time step, Δt, is converted to a head change and is used to calculate new discharges in connected links.

Theory

The flow in a storm drain or sanitary sewer system follows the physical principles of conservation of mass, momentum, and energy. The basic differential equations for sewer flow are obtained from the gradually varied, unsteady flow equations for open channel flow:(4)

$$\frac{\partial Q}{\partial t} = -g \times A \times S_f + 2 \times V \times \frac{\partial A}{\partial t} + V^2 \times \frac{\partial A}{\partial x} - g \times A \times \frac{\partial H}{\partial x} \tag{45}$$

Where:

Q	=	Discharge in pipe.
t	=	Time.
x	=	Longitudinal distance.
V	=	Velocity in pipe.
A	=	Cross-sectional flow area.
H	=	Hydraulic head.
S_f	=	Friction slope.

The friction slope is defined by Manning's equation:

$$S_f = \frac{n^2}{2.22 \times A \times R_h^{\frac{4}{3}}} \times Q \times | V | \qquad (46)$$

Where:

R_h	=	Hydraulic radius.
n	=	Manning's roughness coefficient.

Use of the absolute value sign on the velocity term makes S_f a directional quantity and ensures that the frictional force always opposes the flow. The other quantities are as defined earlier.

HYDRA accounts for junction loss by modifying Manning's n. Sources have recommended the manipulation of Manning's **n** to correct the omission of junction losses. It would be erroneous for a user to estimate a single value of n for a junction because of the dynamic nature of the flows and losses. However, the program incorporates the instantaneous junction loss into Manning's n.(11)

Since the flows coming into a junction are known, the junction loss (H_{jl}) can be determined for that junction (the HGL routine is used here). The pipe friction losses (S_f) can be determined using the original value of Manning's n. These two losses can be combined and a new value of the friction slope, (S^*_f), can be found as shown in equation (47). Next, a new value of n* can be found for the pipe downstream of the junction using equation (48).(15)

$$S_f^* = \frac{S_f L + H_{jl}}{L} \qquad (47)$$

$$n^{*} = \sqrt{\frac{Sf^{*} \times 2.22 A R^{\frac{4}{3}}}{|Q|\,|V|}} \tag{48}$$

This scenario would have the junction loss spread across the length of the outflow pipe, instead of at the junctions. However, the junction elevations would be more accurately predicted than when junction loss is not included. Figure 9 illustrates how HYDRA with the modified **n** hydraulic gradeline would compare to the actual gradeline. One disadvantage to this method is that when the pipe is not flowing full, the water surface in the pipe will be predicted slightly higher than it actually is to account for junction loss. This would tend to add storage to the pipes and attenuate the peak flows. But this would not be so during pressurized flow. The amount of junction loss would not be as great during unpressurized flow; thus the overestimate of the water surface would be small, and therefore should not cause a problem.

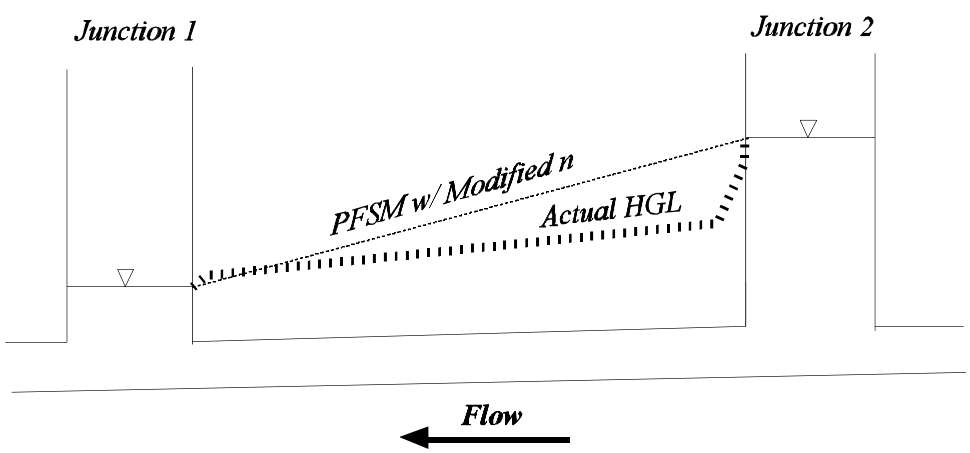

Figure 9. Sample comparison of actual HGL and modified PFSM gradeline.(15)

Considering the entire sewer length as a single computational reach allows derivation of the finite difference form of equation (46). The equation is written in backward time difference between time **n+1**, and time **n**, for the sewer. It is expressed explicitly as:

$$Q_{n+1} = \frac{\left[Q_n + 2 \times \overline{V_n} \times \left(\overline{A_{n+1}} - \overline{A_n}\right) \times \overline{V_n^2} \times \left[\frac{A_{u,n} - A_{d,n}}{L}\right] \times \Delta t - g \times \overline{A_n} \times \left[\frac{H_{u,n} - H_{d,n}}{L}\right] \times \Delta t \right]}{\left[1 + \frac{g \times n^2 \times \Delta t}{2.22 \times \overline{R_h}^{\frac{4}{3}}} \times |\overline{V_n}| \right]} \tag{49}$$

All the symbols above are as previously defined. The subscript **u** denotes the upstream end of a conduit (i.e., entrance) and **d** denotes the downstream end (i.e., exit). The values:

$\overline{V_n}$, $\overline{A_n}$, $\overline{A_{n+1}}$, and $\overline{R_h}$ represent average values at the entrance and exit.

The solution also requires specification of the continuity equation at each junction. The continuity equation for a constant cross-sectional storage junction is written as:

$$\sum Q_i + Q_j = A_j \times \frac{dH}{dt} \tag{50}$$

Where:

Q_i	=	Flow into or out of junction by pipe **i**.
Q_j	=	Time-varied inflow or overflow (outflow) at junction **j**.
A_j	=	Surface area of junction **j**.
H	=	Hydraulic head.

Within the pressure flow simulation, all storage is considered to take place at the junctions. Therefore, water physically located in a pipe must be allocated to the junction at either end. Therefore, the surface area, $\mathbf{A_j}$, of a junction is not the actual surface area of the manhole, **j**, but the surface area of half of the links connected at junction, **j**.

The continuity equation, equation (50), can be expressed in finite difference form in terms of head:

$$H_{n+1} = H_n + \frac{\Delta t}{A_j} \times \left[\Sigma Q_{i,n} + Q_{j,n}\right] \tag{51}$$

Equations (49) and (51) are solved alternately using a modified Euler method. The modified Euler method employs half-step and full-step calculations.

Time Step Selection

Accurate solution of equations (49) and (51) require proper selection of a time step, Δt. From a practical standpoint, pressure flow computations are numerically stable when stability criteria are met for all pipes and junction. The criteria are:

$$Pipe: \quad \Delta t \leq \frac{L}{\sqrt{g \times D}} \tag{52}$$

$$Junction: \quad \Delta t \leq \frac{C' \times A_s \times H_{\max}}{\Sigma Q_i + Q_j} \tag{53}$$

Where:

Δt	=	Time step.
L	=	Pipe length.
C'	=	Dimensionless constant (0.1).
D	=	Pipe diameter.
H_{max}	=	Maximum water-surface level.
A_s	=	Corresponding surface area of junction.
Q_i	=	Flow into or out of junction by pipe **i**.
Q_j	=	Time-varied inflow or overflow (outflow) at junction **j**.
g	=	Acceleration due to gravity.

Examination of equations (52) and (53) reveals that the maximum allowable time step, Δt, is determined by the shortest, smallest pipe having high inflows. In many applications, 15- to 30-s time-steps are numerically adequate. The user must carefully consider the link and junction geometry, the inflow hydrographs, and total simulation time in selecting time steps.

Surcharge and Flooding

The occurrence of surcharge and flooding in a system requires special treatment of the flow and continuity equations. Surcharge occurs when all pipes entering a node are full or when the water surface at the junction lies above the crown of the highest entering pipe, but below the ground surface. Flooding is a special case of surcharge which takes place when the hydraulic gradeline breaks the ground surface and water is lost from the junction to the overlying surface system. When flooding of a junction above the ground surface is detected, the program automatically resets the water surface at the ground elevation. Water rising above this level under flooding conditions is then lost from the system and the pressure flow simulation.

During surcharge, the head calculation in equation (51) is no longer possible because the surface area of the surcharged junction is zero. Thus, the continuity equation becomes:

$$\Sigma Q_i + Q_j = 0 \tag{54}$$

Since flow and continuity are not solved simultaneously in the model, flows computed in the links connected to junction **j** will not satisfy equation (54). By computing $\partial Q_i / \partial H_j$ for each pipe connected to junction **j** and $\partial Q_j / \partial H_j$, a head adjustment can be derived such that the continuity equation is satisfied. Rewriting equation (54) in terms of the adjusted head gives:

$$\sum \left[Q_i + \frac{\partial Q_i}{\partial H_j} \times \Delta H_j \right] + Q_j + \frac{\partial Q_j}{\partial H_j} \times \Delta H_j = 0 \tag{55}$$

By then rearranging terms, the head adjustment can be calculated as follows:

$$\Delta H_j = - \frac{\Sigma Q_i + Q_j}{\Sigma \left(\frac{\partial Q_i}{\partial H_j} \right) + \frac{\partial Q_j}{\partial H_j}} \tag{56}$$

This adjustment is made by half-steps during surcharge through the introduction of an adjustment factor. The model assumes that either the numerical iterations will reach a maximum number set by the user or the algebraic sum of the inflows and outflows of a junction will be less than tolerance. If the former occurs, results should be reviewed for adequacy.

Flow Conditions

The normal flow condition to which the methodology applies is labeled in figure 10. However, it must be modified in other cases. Four special scenarios are illustrated in figure 10.

The normal and special cases are summarized as follows:

1. Normal case - Flow computed from motion equation. Half of flow surface area in pipe assigned to each junction.
2. Critical depth downstream - Lesser of critical and normal depth used downstream. All surface area assigned to upstream junction.
3. Critical depth upstream - Critical depth used. All surface area assigned to downstream junction.
4. Supercritical - Flow set to normal value. Surface area assigned in usual manner as in (1).
5. Dry pipe - Flow set to zero. If any surface area exists, it is assigned to the downstream junction.

Rational Flow Hydrographs

Because the pressure flow module performs a dynamic simulation, the system flows must be dynamic as well. Therefore, for the case of a steady-state flow generated using the Rational method, a time-varied hydrograph must be developed. To develop a hydrograph from the rational peak flow, the following assumptions are made:

1. The rainfall duration, **D**, is equal to the time of concentration, $\mathbf{T_c}$, of the drainage area.
2. The time to peak, $\mathbf{T_p}$, is equal to the time of concentration.
3. The duration of runoff, $\mathbf{T_b}$, is equal to twice the time of concentration.

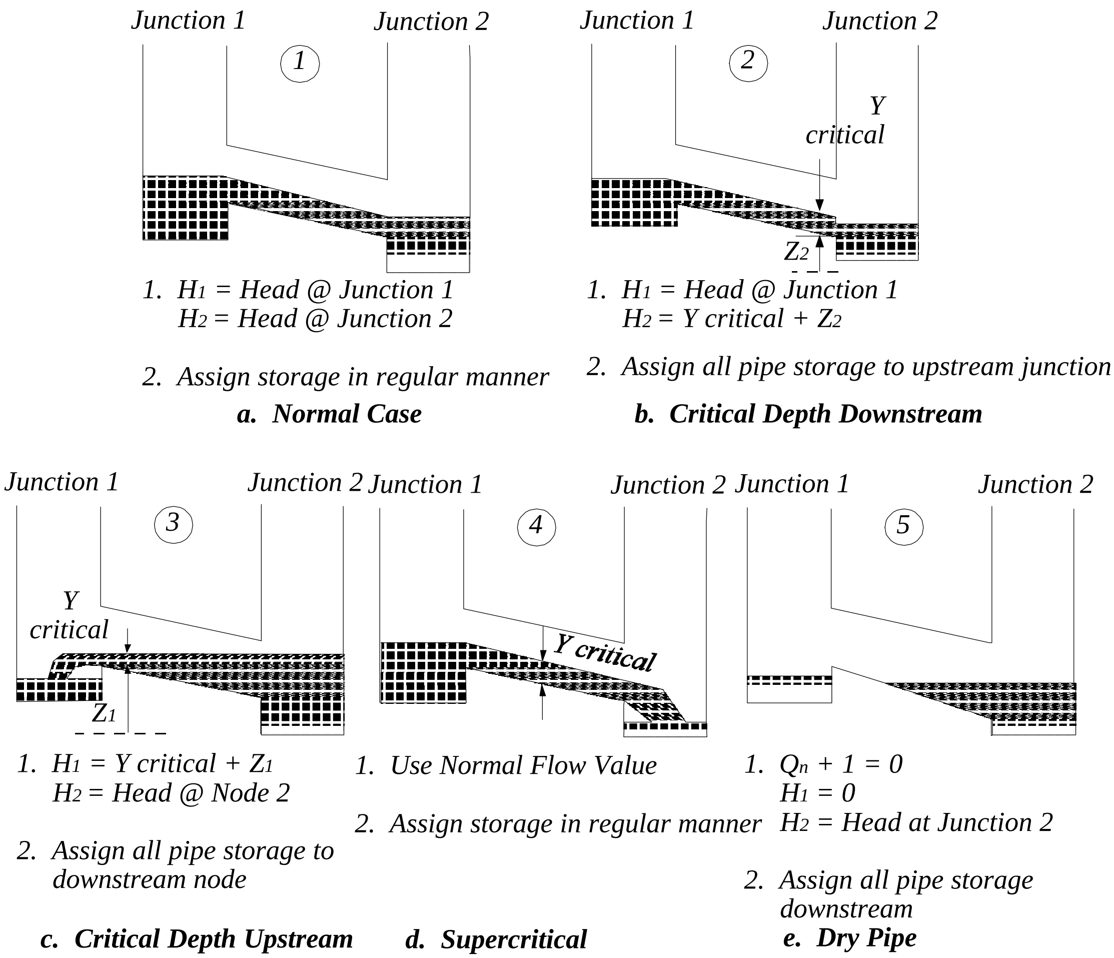

Figure 10. Special flow conditions.

From figure 11, the volume of rainfall excess, as represented by the lower portion of the hyetograph, must equal the volume of the triangular hydrograph. Therefore, CIAD = Q_pT_c and T_c = D; the peak flow, $\mathbf{Q_p}$, of this hydrograph is computed to be equal to CIA.

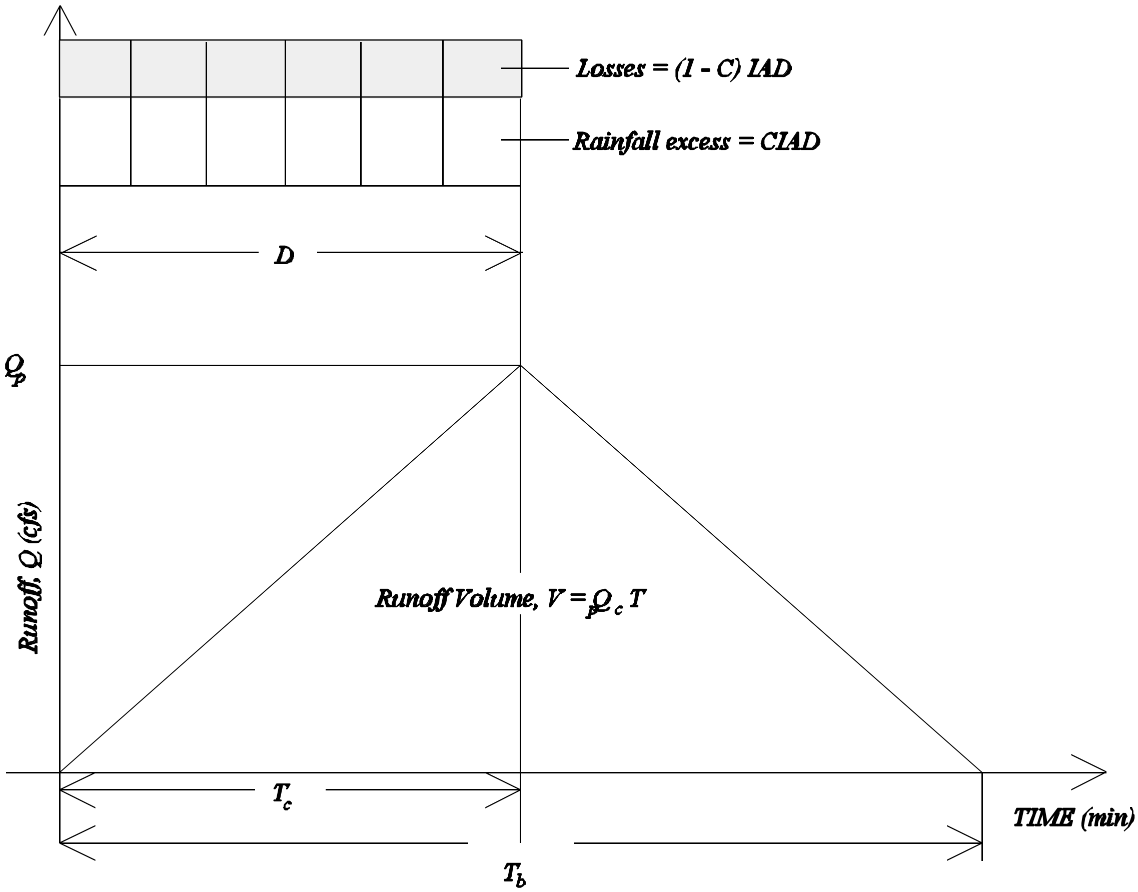

Figure 11. Rational hydrograph.

Limitations of the Pressure Flow Module

The pressure flow module has limitations which, if not appreciated, can result in improperly specified systems and the erroneous computation of heads and flow. The significant limitations are these:

1. Head loss at manholes, expansions, contractions, bends, etc, are estimated based on the hydraulic gradeline method. Minor losses are added by modifying the Manning's **n** specified for the pipes to include both friction and minor loss components.
2. Changes in hydraulic head due to rapid expansions or contractions are neglected. At expansions, the head loss will tend to equalize the heads; but at contractions, the head loss could aggravate the problem.
3. At a manhole where the invert of connecting pipes are different (e.g., a drop manhole), computational errors will occur during surcharge periods if the invert of

the highest pipe lies above the crown of the lowest pipe. The severity of the error increases as the separation increases.

Recommendations for Avoiding Difficulties

Most difficulties in using the Pressure Flow Module arise from three sources:

1. Numerical instability resulting from improper selection of time step and incorrect specification of the total simulation period.
2. Inappropriate iteration control specification.
3. Improper system connectivity.

Numerical stability constraints in the Pressure Flow Module require that **DELT**, the time-step, be no longer than the time it takes flow to travel the length of the shortest pipe in the transport system. A 10-s time-step is recommended for most wet-weather runs, while a 45-s step may be used satisfactorily for dry-weather flow conditions, depending on the system configuration.

Numerical instability in the Pressure Flow Module is signaled by the occurrence of one or more of the following hydraulic conditions:

1. Rapid oscillation of flow and water-surface elevation which is undampened in time: The unstable pipe usually is short relative to other adjacent pipes. The correction is a shorter time-step, a longer pipe length or combination of both. Neither of these should be applied until a careful check of system connections on all sides of the unstable pipe has been made.
2. Excessive velocities (over 6.1 m/s) and discharges which appear to grow without limit at some point in the simulation run: These are manifestations of an unstable pipe element in the system. The cause usually can be traced to the same short pipes or large time steps.
3. A node which continues to “dry up” on each time-step despite a constant or increasing inflow from upstream sources: The cause usually is too large a time-step and excessive discharges in adjacent downstream pipe elements which pull the upstream water surface down. The problem is related to items (1) and (2) and may usually be corrected by a smaller time-step.

4. A large continuity error: A continuity check, which sums the volumes of inflow, outflow, and storage at the beginning and end of the simulation, is found at the end of the intermediate printout. If the continuity error exceeds a reasonable value, the user should check the intermediate printout for pipes with zero flow or oscillating flow. These could be caused by stability or an improperly connected system.

Systems in surcharge may require a special iteration loop, allowing the explicit solution scheme to account for the rapid changes in flows and heads during surcharge conditions. This iteration loop is controlled by two variables on the **PFS** command, **ITMAX**, the maximum number of iterations, and **SURTOL**, a fraction of the flow through the surcharged area. It is recommended that **ITMAX** and **SURTOL** be set initially at 30 and 0.05, respectively. If **ITMAX** is exceeded many times, leaving relatively large flow differentials, the user should increase **ITMAX** to improve the accuracy of the surcharge computation. If, on the other hand, most or all of the iterations do converge, the user may decrease **ITMAX** or increase **SURTOL** to decrease the run time of the model and, consequently, the cost. The user should also keep an eye on the continuity error to ensure that a large loss of water is not caused by the iterations.

In some large systems, more than one area may be in surcharge at the same time. If this occurs and the flows in these areas differ appreciably, those areas with the smallest flows may not converge, while areas with large flows will. This is because both the tolerance and flow differential are computed as sums of all flows in surcharge. It is possible, therefore, for overall convergence to occur even when relatively large flow errors still exist in surcharge areas with small flows. To correct this, **SURTOL** can be decreased until the flow differential for the area in question decreases to a small value over time. It should be noted, however, that large flow differentials for a short period of time are not unusual providing they decrease to near or below the established tolerance for most of the simulation.

Prior to a lengthy run of the Pressure Flow Module for a new system, a short test run should be made to confirm that the link-node model is properly connected and correctly represents the prototype. This check should be made on the echo of the input data, which shows the connecting links at each junction. The geometric/hydraulic data for each pipe and junction should also be confirmed. Particular attention should be paid to the junction location of outfalls to ensure these conform to the prototype system. Using this short test run, the total number of pipes and junctions, including internal links and nodes created, can be determined from the junctions and pipe characteristic tables in the output. The length of all pipes in the system should be consistent. If possible, each pipe should be at least 30.5 m in length. This constraint may be difficult to meet in the vicinity of abrupt changes in pipe configurations which must be represented in the model. However, the length of the shortest pipe does directly determine the maximum time step.

SUMMARY

Methodologies used for the key features of HYDRA have been described including techniques for flow generation, flow conveyance, inlet computations, storage, cost estimation, hydraulic gradeline computations and pressure flow simulations. Not all of these methodologies will be used in any given design or analysis. However, it is important for the user to understand the methodologies applied in a given situation to ensure that appropriate application of the techniques is achieved.

USER DOCUMENTATION

Effective use of HYDRA requires an understanding of the interaction between the user and the software. While the previous chapter describes what HYDRA does, this chapter explains how one communicates with the software to achieve desired results.

THE COMMAND APPROACH - ORGANIZING THE DATA

HYDRA operates through the command language concept. This means that data entry and data analysis are all dictated by user-supplied commands. A command is a very specific entity that describes one basic task that HYDRA can recognize. There is only a set number of commands in HYDRA's vocabulary and each must follow a specific format. Currently there are 56 commands that HYDRA can recognize, although this number is subject to change as long as improvements are being added to HYDRA. A complete list, to date, of these commands along with brief definitions, is shown in table 3. Appendix C includes a more detailed description, including format specifications.

Table 3. Glossary of commands.

Command		Description
BEN	-	specifies Pipe **BEN**d data such as angle and radius.
BOX	-	rectangular pipe that transport system flow from one point to another.
CHA	-	allows definition of an open **CHA**nnel or ditch.
CRI	-	determines whether inverts or crowns are to be matched (**CRI**teria).
CST	-	sets geometry factors and unit prices (**C**o**ST**s in place).
DIV	-	splits the system flow into two components (**DIV**erts flow).
ECF	-	allows **E**xtra **C**osts per linear meter to be added to the pipe cost.
EFF	-	describes inlet performance in ordered pairs of flow versus **EFF**iciency.
ELP	-	horizontal or vertical **El**li**P**se that transports system flow.
END	-	**END**s a command string.
EXC	-	establishes trench **EXC**avation costs.
FLO	-	adds or subtracts a constant **FLO**w to the system.
GET	-	**GET**s a gutter hydrograph from storage.
GPC	-	sets the **G**allons **P**er **C**apita per day for flow calculations.
GUT	-	establishes **GUT**ter characteristics.
HGL	-	signals that **H**ydraulic **G**rade**L**ine computations should be made.
HOL	-	**HOL**ds system flow at the lower end of a lateral.
IDY	**-**	**I**nitializes **D**epths for pressure flow.

Table 3. Glossary of commands (continued).

Command		Description
INF	-	inputs **INF**iltration flows by population or area.
INL	-	sets parameters for a storm water **INL**et.
IPU	-	establishes the number of **I**ndividuals **P**er sanitary **U**nit.
IQV	-	**I**nitializes flows and **V**elocities.
JOB	-	initiates **JOB** and enters **JOB** title.
LOS	-	allows input of additional pipe **LOS**ses.
LPC	-	calculates and **L**ists **P**ipe **C**osts in place.
MAP	-	establishes factor for converting in^2 on a **MAP** to ac.
NEW	-	clears some registers and loads **NEW** lateral name.
PCF	-	establishes a **P**ipe **C**ost **F**actor.
PCO	-	establishes **P**ipe **CO**sts per meter in place.
PDA	-	establishes **P**ipe design **DA**ta.
PEA	-	translates average daily flow into **PEA**k daily flow.
PFS	-	specifies **P**ressure **F**low **S**imulation parameters.
PFP	-	**P**rints **F**low in **P**ipes for pressure flow simulation.
PHJ	-	**P**rints **H**ead of **J**unctions for pressure flow simulation.
PIP	-	moves water from one point to another in a circular **PIP**e.
PNC	-	specifies **P**ipe-**N**ode **C**onnections for hydraulic gradeline computation.
PON	-	allows surface **PON**ding of flows **HYD**, **GUT**, **GET** commands.
PSZ	-	sets **P**ipe **S**i**Z**es for design.
PUM	-	lifts the hydraulic gradient a specified amount (**PUM**p).
PUT	-	**PUT**s gutter flow into storage.
RAI	-	sets the values on a **RAI**nfall intensity versus duration curve.
REC	-	**REC**alls flow previously stored using the **HOL** or **DIV** commands.
REM	-	allows a line for **REM**arks or comments.
RES	-	allows the analysis of in-line storage (**RES**ervoir).
SAF	-	applies **SAF**ety factors to calculated flows.
SAN	-	enters **SAN**itary flow into the system.
SDI	-	allows input of **S**tage-**DI**scharge curve.
SST	-	allows input of **S**tage-**ST**orage curve.
STE	-	sets the length of time increments (**STE**ps).
STO	-	enters subbasin data for determining **STO**rm water design flow.
SUN	-	enters the number of contributing **S**anitary **UN**its.
SWI	-	sets **SWI**tch for determining method of storm/sanitary flow analysis.
TRA	-	**TRA**nsfer system flow to surface flow.
TSL	-	determines **T**rench side wall **SL**ope.
TWE	-	allows for the input of a **T**ail**W**ater **E**levation at the system outfall.
UHY	-	enables use of externally produced hydrographs (**U**ser **HY**drograph).

Commands are the data that a user must specify to describe a system for analysis. These commands may be arranged in almost any order, provided they follow a few, simple guidelines. These guidelines ensure that the users system is described appropriately and logically, and will become more clear as the user gains familiarity with this section and the examples provided in the appendix. Once these commands are arranged in their final working order, they are collectively referred to as a command string. The command string is what HYDRA needs to define a system model for analysis.

Figure 12 shows an example command string, broken down into its command name and accompanying data field, with an explanation of each command used. Ordering commands in a HYDRA command string is a relatively easily acquired skill. For instance, using figure 12 as an example, note that the **JOB** command is first. This establishes the file name that will be used for the output. The next command is the **PDA** command which establishes certain pipe data criteria such as the Manning's "n" friction factor, minimum diameter, ideal depth, minimum cover, minimum velocity, and minimum slope. The **PDA** command does not need to be the second command, but if it is used, it must precede the first **PIP** (pipe) command so that HYDRA will have the criteria necessary to formulate a preliminary pipe design. The **PIP** command is the third command in the data set.

HYDRA Command	Data	Comments
JOB	Command Example	The title of the HYDRA job.
PDA	0.014 300 2.5 1.5 0.5 0.005	Sets initial design criteria, such as Manning's **n**, minimum diameter, etc.
NEW	Main Street	Starts a new lateral, providing it with a unique name.
FLO	0.500	Places a 0.500m^3/s FLOw into the system at this location.
PIP	100 33.500	Pipe parameters. (Transports the flow in a pipe 100-m long **-** ground elevation at both the upper and lower ends is 33.500 m.)
PIP	120 33.500 34.000	Continues to transport the flow into another pipe 120-m long. Notice that the ground elevation at the upstream end is 33.500 m and 34.000 m at the downstream end. HYDRA can model adverse ground elevations.
FLO	0.100	Adds another 0.100 m^3/s to the system at this point.
PDA	0.025 300 2.5 1.5 0.5 0.005	Change the pipe friction factor to 0.025 (corrugated metal pipe).
PIP	140 34.000 30.500	Transports the flow another 140 m.
END		Terminates the HYDRA run and prints the results.

Figure 12. Example of a command string.

In most cases, commands may be used more than once. This allows the user to change specifications or design criteria at any point in the system. For example, the **PDA** command was used twice. Although this example shows modification only to the friction factor, any parameter value may be modified by changing the appropriate subfield in the data field.

Figure 13 shows the organization of all the HYDRA commands. Although the rules for ordering commands are not listed in this figure, the reader should grasp that HYDRA command strings are assembled according to the loose hierarchy of figure 13. Commands operate in "free format" fashion; that is, a space [ ], or comma [,] are parameter subfield separators that may be used in any amount between each parameter value (spacing between subfields is not critical). Continuation of a command with many or extremely lengthy variables is achieved by simply continuing the data onto the next line.

THE HYDRAIN ENVIRONMENT

For those users who have obtained HYDRA as part of the Federal Highway Administration's **HYDRAIN** package, consult the HYDRAIN documentation for information on how to use the software system.

There are three methods to execute the HYDRA program. Two of these are within the HYDRAIN environment. The first method is to run HYDRA from the HYDRAIN editor. This allows the user the option of immediate review and editing capabilities. The second method is to run HYDRA from the HYDRAIN shell using the **Analyze** option. The final method is to execute HYDRA from the DOS prompt. Detailed discussion of all three of these options is in the HYDRAIN portion of the documentation.

Upon completion of the run, HYDRA assigns the output file and an **.LST** filename extension. The output file contains an echo of the input data and the results. The output may contain message describing warnings or errors encountered during execution. These messages are useful in debugging an input data set.

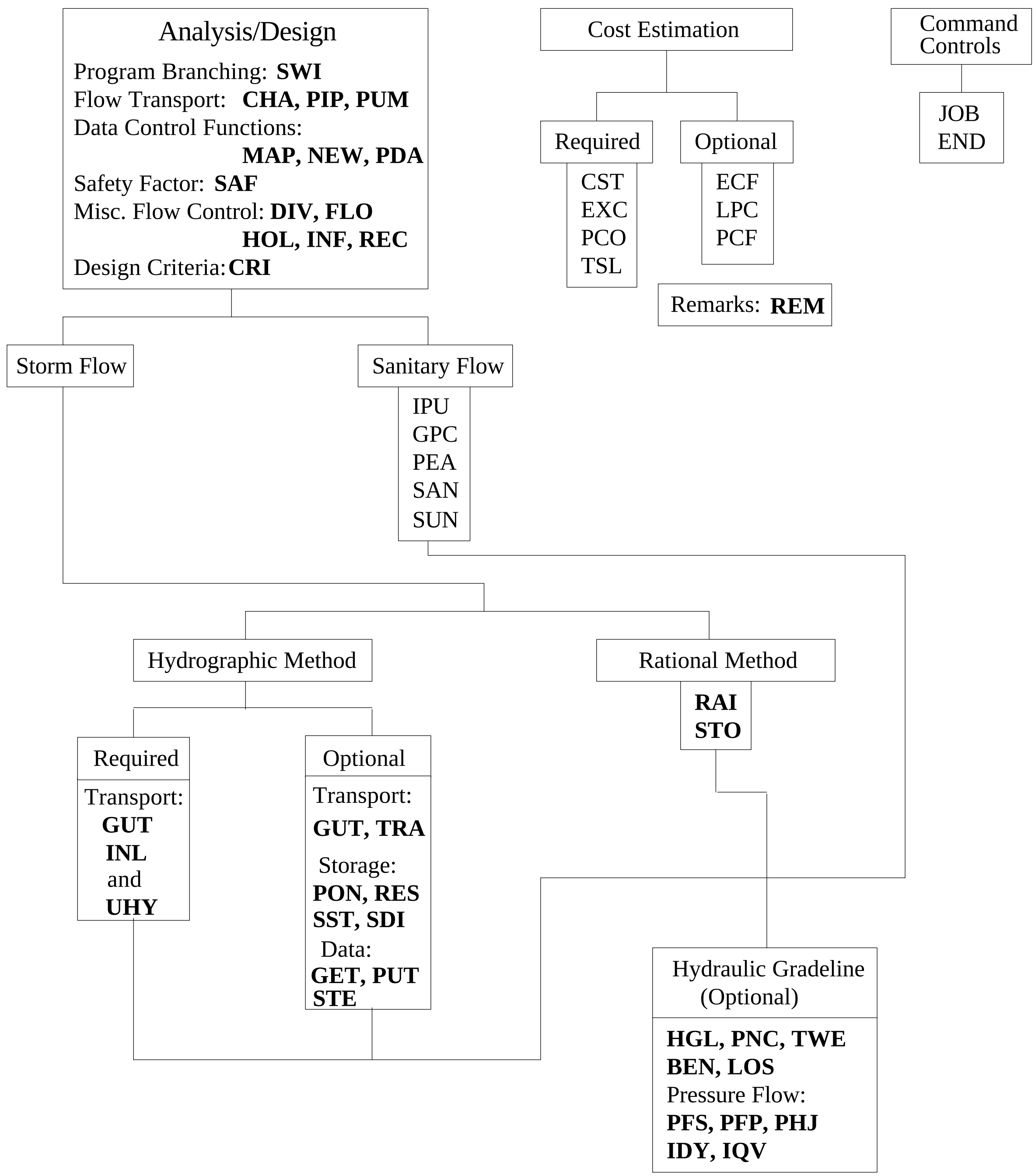

Figure 13. Organization of HYDRA commands.

APPENDIX A: BENCHMARK EXAMPLES

The following examples are hypothetical systems modeled by HYDRA which are provided to illustrate some of the program's capabilities. It should be recognized that these examples are not meant to give a comprehensive guide of every command option. The user is referred to appendix C for this information. It is intended that these examples will achieve at least these four objectives:

1. Provide guidance for creating command strings.
2. Demonstrate uses for many of the commands.
3. Provide information on how to set up a problem.
4. Demonstrate what to expect for output.

The examples offered here collectively make use of most of the available commands. In each case, a figure is included to schematically represent a given problem. Following each figure, the input data set for the run and its corresponding output are given. Each of the seven examples provides a different type of application of HYDRA. These are:

1. Rational method storm drain design with the hydraulic gradeline.
2. Rational method analysis with pressure flow.
3. Hydrographic simulation storm drain analysis/design.
4. Hydrographic analysis and pressure flow simulation.
5. Sanitary sewer design.
6. Sanitary sewer analysis.
7. Combined system analysis.

Example One: Rational Method Design with Hydraulic Gradeline

Problem:

The example shown in figure 14 illustrates the use of HYDRA for design of a storm drain using the Rational method. As for any design problem, the cost commands (EXC, TSL, PCO, and CST) must be used if cost estimates are desired. Also, a PDA command must be provided to inform HYDRA what pipe characteristics are desired. This command can be used at least once. Four additional commands are introduced in this example: STO, RAI, HGL, and PNC. The STO command provides the physical characteristics of a drainage area required for the Rational method: size, runoff coefficient, and time of concentration. The RAI command supplies the Intensity-Duration-Frequency (IDF) curve for the return period and location the user desires to analyze. HYDRA uses the time of concentration for each individual area and then consults the IDF curve to find an intensity for the rational formula. As HYDRA proceeds through the system, it continually adjusts the time of concentration as more areas are being aggregated. The HGL and PNC commands are related to the calculation of the hydraulic gradeline through the system. The HGL commands acts as a switch to “turn on” the computations. The PNC commands details the pipe-node connectivity through the system, as well as the other parameters required to calculate minor losses.

Data file: FOXHALL.IDF

```
FOXHALL CRESCENTS INTENSITY-DURATION-FREQUENCY
    8
         5    192.02
        10    160.02
        15    138.18
        30    100.33
        60     67.56
       100     48.77
       120     43.18
       360     19.81
```

Input file: HYDRA1.HDA

```
JOB Example One: Rational Method Design with Hydraulic Gradeline
SWI 2
PDA 0.013 300 1.22 0.914 0.61 0.001
RAI FOXHALL.IDF
EXC 1.52 0.94 7.62 1.48
TSL 0 0.25 3.05 0.15
PCO 300 15.03 900 81.50
CST 1.5 4.92 0 0 0 1.64 8.12 1 0 1.99 1.05 0.79 2.04 1.99 0
HGL 1
REM Begin Main Lateral
NEW MAIN STREET
REM Main Lateral: Junctions 1 to 2
STO 2.10 0.2 21
PIP 137.15 58.98 58.52
PNC 1 2 0.914 125 0 1
REM Main Lateral: Junctions 2 to 3
STO 1.62 0.5 28
STO 2.35 0.6 15
STO 1.05 0.6 15
PIP 228.59 58.52 56.17
PNC 2 3 1.52 180 0 0
REM Main Lateral: Junctions 3 to 7
```

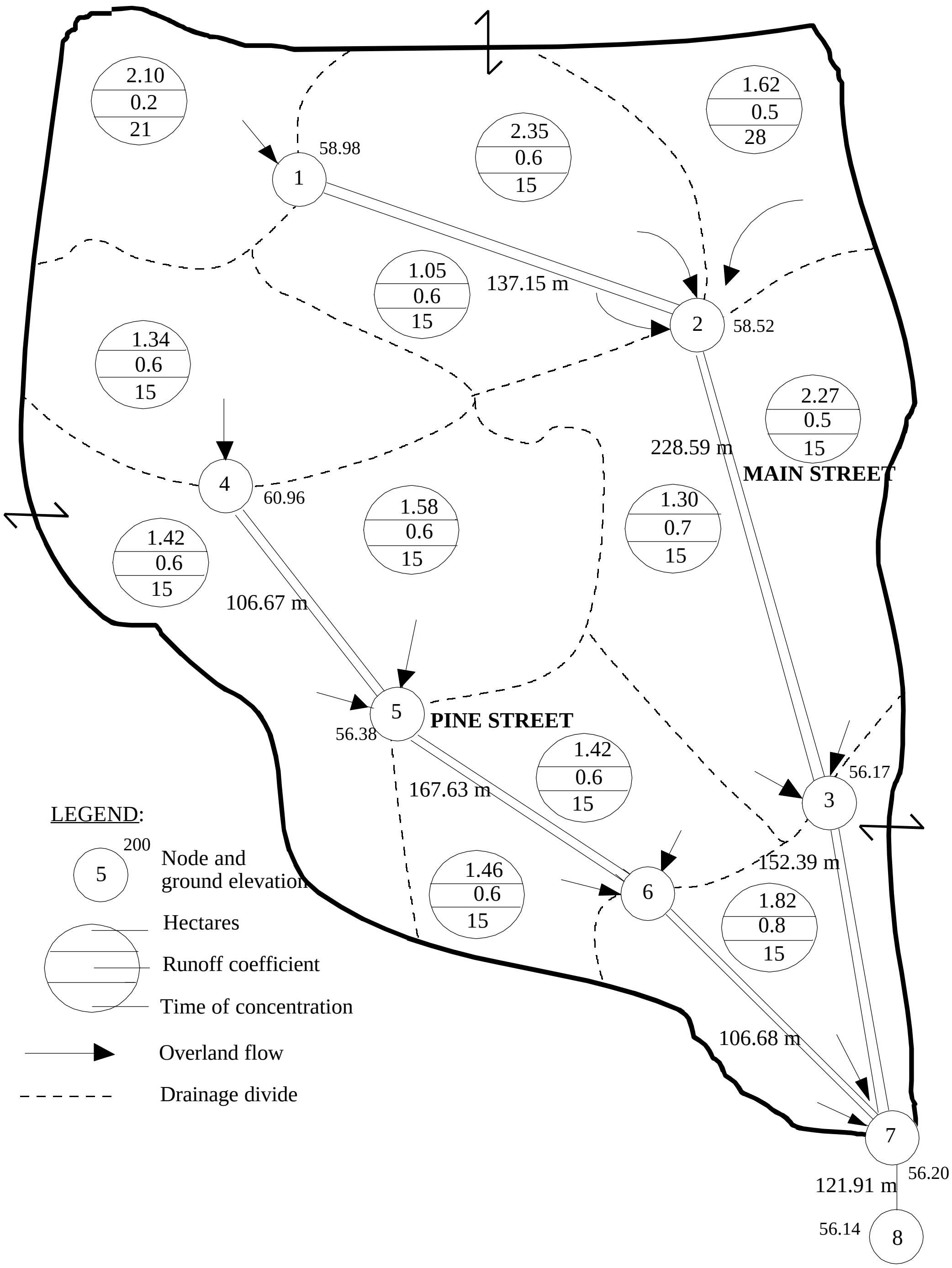

Figure 14. Rational method design.

```
STO 2.27 0.5 15
STO 1.30 0.7 15
PIP 152.39 56.17 56.20
PNC 3 7 2.13 180 0 3
HOL 1
REM Begin Second Lateral
NEW PINE STREET
REM Side Lateral: Junctions 4 to 5
STO 1.34 0.6 15
PIP 106.67 60.96 56.38
PNC 4 5 1.22 155 0 1
REM Side Lateral: Junctions 5 to 6
STO 1.42 0.6 15
STO 1.58 0.6 15
PIP 167.63 56.38 56.20
PNC 5 6 1.52 180 0 2
REM Side Lateral: Junctions 6 to 7
STO 1.46 0.6 15
STO 1.42 0.6 15
PIP 106.68 56.17 56.20
PNC 6 7 2.13 150 0 3
REC 1
STO 1.82 0.8 15
REM Outfall Link: Junctions 7 to 8
PIP 121.91 56.20 56.14
PNC 7 8 0 180 2
END
```

Discussion of output:

The gravity flow simulation portion of the results provides tables of designed pipe sizes, invert elevations, and costs. This pipe design scenario is then “sent” to the hydraulic gradeline component of the model. The hydraulic gradeline table indicates the pipe links that could possibly surcharge. However, since HGL is based on pipe full flow, the occurrence of super-critical flow in a surcharged pipe would be contradictory to the surcharged condition. In this case, it would be recommended to run the pressure flow simulation option.

Output file: HYDRA1.LST

```
         ******  HYDRA  ******* (Version 6.1) *****         Date 07-16-1998
                                                               Page No   1


  Example One: Rational Method Design with Hydraulic Gradeline


+++ Commands Read From File C:\HYDRA\HYDRA1.HDA
  JOB
  SWI 2
  PDA 0.013 300 1.22 0.914 0.61 0.001
  RAI FOXHALL.IDF
+++ Notice:  Intermediate file has data in SI units
```

IDF CURVE

```
192.*))))))))))))))))))3))))))))))))))))))3))))))))))))))))))3)))))))))),
         *                  .                   .                   .
*
 R       *                  .                   .                   .
*
 a       **                 .                   .                   .
*
 i   150.3                  .                   .                   .
3
 n       * *                .                   .                   .
*
 f       *                  .                   .                   .
*
 a       *                  .                   .                   .
*
 l       *                  .                   .                   .
*
 l   100.3    *             .                   .                   .
3
         *                  .                   .                   .
*
 i       *                  .                   .                   .
*
 n       *          *       .                   .                   .
*
         *                  .                   .                   .
*
 m    50.3                  *                   .                   .
3
 m       *                  .   *               .                   .
*
 /       *                  .                   .                   .
*
 h       *                  .                   .                   .
*
         *                  .                   .                   .
*

0.*))))))))))))))))))3))))))))))))))))))3))))))))))))))))))3))))))))))-
       5.                100.                200.                300.
360.
```

Duration, t (min)

PLOT-DATA (Time, t(min) vs. Intensity, i(mm/h))

t	i	t	i	t	i	t	i	t	i
5.	192.	60.	68.	0.	0.	0.	0.	0.	0.
10.	160.	100.	49.	0.	0.	0.	0.	0.	0.
15.	138.	120.	43.	0.	0.	0.	0.	0.	0.
30.	100.	360.	20.	0.	0.	0.	0.	0.	0.

```
EXC 1.52 0.94 7.62 1.48
TSL 0 0.25 3.05 0.15
PCO 300 15.03 900 81.50
CST 1.5 4.92 0 0 0 1.64 8.12 1 0 1.99 1.05 0.79 2.04 1.99 0
HGL 1
```

```
  Example One: Rational Method Design with Hydraulic Gradeline


  REM Begin Main Lateral
  NEW MAIN STREET
  REM Main Lateral: Junctions 1 to 2
  STO 2.10 0.2 21
  PIP 137.15 58.98 58.52
+++ Tc =  21.0 min
+++ CA =    .4
+++ Top width of trench ---> UP  1.5 m
                        ---> DN  1.5 m
+++ Link #  1,  Flow depth =    .324 m, Critical depth =    .264 m
  PNC 1 2 0.914 125 0 1
  REM Main Lateral: Junctions 2 to 3
  STO 1.62 0.5 28
  STO 2.35 0.6 15
  STO 1.05 0.6 15
  PIP 228.59 58.52 56.17
+++ Tc =  28.0 min
+++ CA =   3.3
*** WARNING: Pipe invert at D/S end dropped to meet minimum depth
    to invert criterion at D/S end
*** WARNING: Pipe invert at U/S dropped to meet cover criterion at U/S end
*** WARNING: Inverts at D/S & U/S dropped to meet cover criterion at D/S end
+++ Top width of trench ---> UP  2.0 m
                        ---> DN  1.9 m
+++ Link #  2,  Flow depth =    .547 m, Critical depth =    .602 m
  PNC 2 3 1.52 180 0 0
  REM Main Lateral: Junctions 3 to 7
  STO 2.27 0.5 15
  STO 1.30 0.7 15
  PIP 152.39 56.17 56.20
+++ Tc =  29.4 min
+++ CA =   5.3
*** WARNING: Pipe invert at U/S dropped to meet cover criterion at U/S end
+++ Top width of trench ---> UP  2.8 m
                        ---> DN  2.8 m
+++ Link #  3,  Flow depth =    .999 m, Critical depth =    .645 m
  PNC 3 7 2.13 180 0 3
  HOL 1
  REM Begin Second Lateral
  NEW PINE STREET
  REM Side Lateral: Junctions 4 to 5
  STO 1.34 0.6 15
  PIP 106.67 60.96 56.38
+++ Tc =  15.0 min
+++ CA =    .8
+++ Top width of trench ---> UP  1.4 m
                        ---> DN  1.4 m
+++ Link #  4,  Flow depth =    .266 m, Critical depth =    .362 m
  PNC 4 5 1.22 155 0 1
```

```
  Example One: Rational Method Design with Hydraulic Gradeline

  REM Side Lateral: Junctions 5 to 6
  STO 1.42 0.6 15
  STO 1.58 0.6 15
  PIP 167.63 56.38 56.20
+++ Tc =  15.5 min
+++ CA =   2.6
*** WARNING: Pipe invert at U/S dropped to meet cover criterion at U/S end
*** WARNING: Inverts at D/S & U/S dropped to meet cover criterion at D/S end
+++ Top width of trench ---> UP  2.6 m
                        ---> DN  2.6 m
+++ Link #  5,  Flow depth =    .816 m, Critical depth =    .537 m
  PNC 5 6 1.52 180 0 2
  REM Side Lateral: Junctions 6 to 7
  STO 1.46 0.6 15
  STO 1.42 0.6 15
  PIP 106.68 56.17 56.20
+++ Tc =  17.8 min
+++ CA =   4.3
*** WARNING: Pipe invert at U/S dropped to meet cover criterion at U/S end
+++ Top width of trench ---> UP  2.8 m
                        ---> DN  2.8 m
+++ Link #  6,  Flow depth =   1.040 m, Critical depth =    .661 m
  PNC 6 7 2.13 150 0 3
  REC 1
  STO 1.82 0.8 15
  REM Outfall Link: Junctions 7 to 8
  PIP 121.91 56.20 56.14
+++ Tc =  31.3 min
+++ CA =  11.1
*** WARNING: Pipe invert at U/S dropped to meet cover criterion at U/S end
+++ Top width of trench ---> UP  3.4 m
                        ---> DN  3.5 m
+++ Link #  7,  Flow depth =   1.260 m, Critical depth =    .852 m
  PNC 7 8 0 180 2
  END
 END OF INPUT DATA.
```

Example One: Rational Method Design with Hydraulic Gradeline

*** MAIN STREET — Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
1	137	450	57.579	.003	1.402	.914	1.166	.142	5206.
			57.118		1.402		1.038	.165	
2	229	750	56.612	.009	1.908	.914	2.763	.949	17385.
			54.444		1.726		2.455	1.084	
3	152	1350	53.793	.001	2.377	.914	1.330	1.492	23687.
			53.641		2.559		1.179	1.688	

Length = 518. m Total length = 518. m
Cost = 46279. Total Cost = 46279.

*** PINE STREET — Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
4	107	375	59.640	.043	1.320	.914	3.687	.306	3061.
			55.060		1.320		3.290	.363	
5	168	1200	54.154	.001	2.226	.914	1.209	.983	22588.
			53.986		2.214		1.090	1.233	
6	107	1350	53.793	.001	2.377	.914	1.338	1.565	16570.
			53.687		2.513		1.179	1.688	
7	122	1800	53.336	.001	2.864	.914	1.597	3.026	26560.
			53.214		2.926		1.429	3.636	

Length = 503. m Total length = 1021. m
Cost = 68780. Total Cost = 115058.

Example One: Rational Method Design with Hydraulic Gradeline

Hydraulic Gradeline Computations

Link #	Down-stream Node #	Hydraulic Gradeline Elevation	Crown Elev.	Possible Surcharge	Ground Elev.	Super-crit.?	Manhole Depth	Loss Coef
1	2	57.569	57.576	N	58.520	N	.957	.28
2	3	55.046	55.206	N	56.170	Y	1.090	.01
3	7	54.726	54.987	N	56.200	N	1.390	.08
4	5	55.422	55.441	N	56.380	Y	.891	.01
5	6	54.925	55.180	N	56.170	N	1.132	.07
6	7	54.726	55.033	N	56.200	N	1.390	.09
7	8	54.474	55.017	N	56.140	N	1.260	.00

Link #	Terminal Node #	Hydraulic Gradeline Elevation	Ground Elevation	Loss Coef.
1	1	58.007	58.980	1.50
4	4	60.946	60.960	1.50

NORMAL END OF HYDRA

Example Two: Rational Method Analysis with Pressure Flow

Problem:

HYDRA is a useful tool in analyzing existing systems. This example, shown in figure 14, demonstrates the use of HYDRA to analyze a storm drain system using the Rational method. For this example, the system suggested by HYDRA in example one is entered in the command string as an existing system and analyzed. (Note the extra parameters on the PIP commands.) The system is identical to that designed in example one except that a higher return period is employed (the intensity/duration values are hand entered). Note that the RAI command requires two lines. Continuation of a line is achieved by making the first three columns in the second line blank.

The commands that are introduced in this example are: **PFS**, **PFP**, **PHJ**, and **TWE**. The **PFS** command initiates the pressurized flow simulation by establishing the control parameters. Commands **PFP** and **PHJ** select the pipes and junctions, respectively, to be printed out for the detailed printout. The **TWE** command specifies the tailwater elevation at the outfall of the system which is necessary for the pressure flow simulation. The default tailwater elevation is the invert of the outfall pipe (in this example, 53.35 m). Just like the hydraulic gradeline computations, the **PNC** and **PIP** commands define the pipe-node connectivity for the entire system.

Input file: HYDRA2.HDA

```
JOB Example Two: Rational Method Analysis with Pressure Flow
SWI 2
PDA 0.013 300 1.22 0.914 0.61 0.001
RAI 5,223.77  10,197.61   15,168.91   30,140.72   60,102.87
    90,86.61   120,84.33   360,53.59
TWE 53.35
REM Begin Main Lateral
NEW MAIN STREET
REM Main Lateral: Junctions 1 to 2
STO 2.10 0.2 21
PIP 137.15 58.98 58.52 57.57 57.11 -450
PNC 1 2 0.914 125 0 1
REM Main Lateral: Junctions 2 to 3
STO 1.62 0.5 28
STO 2.35 0.6 15
STO 1.05 0.6 15
PIP 228.59 58.52 56.17 56.59 54.43 -750
PNC 2 3 1.52 180 0 0
REM Main Lateral: Junctions 3 to 7
STO 2.27 0.5 15
STO 10.50 0.7 15
PIP 152.39 56.17 56.20 53.77 53.62 -1350
PNC 3 7 2.13 180 0 3
HOL 1
REM Begin Second Lateral
NEW PINE STREET
REM Side Lateral: Junctions 4 to 5
STO 1.34 0.6 15
PIP 106.68 60.96 56.38 59.63 55.06 -375
PNC 4 5 1.22 155 0 1
REM Side Lateral: Junctions 5 to 6
STO 1.42 0.6 15
```

```
STO 1.58 0.6 15
PIP 167.63 56.38 56.20 54.22 54.05 -1200
PNC 5 6 1.52 180 0 2
REM Side Lateral: Junctions 6 to 7
STO 1.46 0.6 15
STO 1.42 0.6 15
PIP 106.67 56.17 56.20 53.77 53.66 -1350
PNC 6 7 2.13 150 0 3
REC 1
STO 1.82 0.8 15
REM Outfall Link: Junctions 7 to 8
PIP 121.91 56.20 56.14 53.31 53.18 -1800
PNC 7 8 0 180 2
PFS 0 60 10 10 1 3 2 30 0.05
PHJ 2 3 6
PFP 3 7
END
```

Discussion of output:

In the results of example two, the gravity flow simulation does suggest the possibility of surcharge. The gravity flow module notices which pipes have a design flow larger than the normal pipe capacity. The module then provides two alternatives (removing excess flow or adding additional pipe capacity) that would return the system to free surface flow conditions. In reality, the pipes would act under pressure. The second (and more substantive) output portion depicts results of the pressure flow simulation. Note that the pipe junction connectivity is reiterated, which is useful for debugging the network. Since the network in this example is the exact same as the previous example, the surcharging of the pressure flow simulation can be contrasted to that of the earlier HGL simulation.

Output file: HYDRA2.LST

```
          ******  HYDRA  ******* (Version 6.1) *****         Date 07-16-1998
                                                                Page No   1

   Example Two: Rational Method Analysis with Pressure Flow


 +++ Commands Read From File C:\HYDRA\HYDRA2.HDA
   JOB
   SWI 2
   PDA 0.013 300 1.22 0.914 0.61 0.001
   RAI 5,223.77  10,197.61   15,168.91   30,140.72   60,102.87
    90,86.61   120,84.33   360,53.59
```

```
                         IDF CURVE

     224.*))))))))))))))))))3)))))))))))))))))))3)))))))))))))))))))3))))))))))),
         *                  .                   .                   .          *
 R       **                 .                   .                   .          *
 a   200.3                  .                   .                   .          3
 i       *                  .                   .                   .          *
 n       * *                .                   .                   .          *
 f       *                  .                   .                   .          *
 a   150.3    *             .                   .                   .          3
 l       *                  .                   .                   .          *
 l       *                  .                   .                   .          *
         *          *       .                   .                   .          *
 i   100.3                  .                   .                   .          3
 n       *                * .    *              .                   .          *
         *                  .                   .                   .          *
 m       *                  .                   .                   .          *
 m    50.3                  .                   .                   .          3
 /       *                  .                   .                   .          *
 h       *                  .                   .                   .          *
         *                  .                   .                   .          *
       0.*))))))))))))))))))3)))))))))))))))))))3)))))))))))))))))))3)))))))))))-
       5.                100.                200.                300.       360.

                         Duration, t (min)
```

PLOT-DATA (Time, t(min) vs. Intensity, i(mm/h))

t	i	t	i	t	i	t	i	t	i
5.	224.	60.	103.	0.	0.	0.	0.	0.	0.
10.	198.	90.	87.	0.	0.	0.	0.	0.	0.
15.	169.	120.	84.	0.	0.	0.	0.	0.	0.
30.	141.	360.	54.	0.	0.	0.	0.	0.	0.

```
TWE 53.35
REM Begin Main Lateral
NEW MAIN STREET
REM Main Lateral: Junctions 1 to 2
```

```
  Example Two: Rational Method Analysis with Pressure Flow

  STO 2.10 0.2 21
  PIP 137.15 58.98 58.52 57.57 57.11 -450
+++ Tc =  21.0 min
+++ CA =    .4
+++ Link #  1,  Flow depth =    .450 m, Critical depth =    .450 m
  PNC 1 2 0.914 125 0 1
  REM Main Lateral: Junctions 2 to 3
  STO 1.62 0.5 28
  STO 2.35 0.6 15
  STO 1.05 0.6 15
  PIP 228.59 58.52 56.17 56.59 54.43 -750
+++ Tc =  28.0 min
+++ CA =   3.3
+++ Link #  2,  Flow depth =    .750 m, Critical depth =    .750 m
  PNC 2 3 1.52 180 0 0
  REM Main Lateral: Junctions 3 to 7
  STO 2.27 0.5 15
  STO 10.50 0.7 15
  PIP 152.39 56.17 56.20 53.77 53.62 -1350
+++ Tc =  29.3 min
+++ CA =  11.8
+++ Link #  3,  Flow depth =   1.350 m, Critical depth =   1.350 m
  PNC 3 7 2.13 180 0 3
  HOL 1
  REM Begin Second Lateral
  NEW PINE STREET
  REM Side Lateral: Junctions 4 to 5
  STO 1.34 0.6 15
  PIP 106.68 60.96 56.38 59.63 55.06 -375
+++ Tc =  15.0 min
+++ CA =    .8
+++ Link #  4,  Flow depth =    .375 m, Critical depth =    .375 m
  PNC 4 5 1.22 155 0 1
  REM Side Lateral: Junctions 5 to 6
  STO 1.42 0.6 15
  STO 1.58 0.6 15
  PIP 167.63 56.38 56.20 54.22 54.05 -1200
+++ Tc =  15.5 min
+++ CA =   2.6
+++ Link #  5,  Flow depth =    .960 m, Critical depth =    .596 m
  PNC 5 6 1.52 180 0 2
  REM Side Lateral: Junctions 6 to 7
  STO 1.46 0.6 15
  STO 1.42 0.6 15
  PIP 106.67 56.17 56.20 53.77 53.66 -1350
+++ Tc =  17.8 min
+++ CA =   4.3
+++ Link #  6,  Flow depth =   1.350 m, Critical depth =   1.350 m
  PNC 6 7 2.13 150 0 3
```

```
  Example Two: Rational Method Analysis with Pressure Flow

  REC 1
  STO 1.82 0.8 15
  REM Outfall Link: Junctions 7 to 8
  PIP 121.91 56.20 56.14 53.31 53.18 -1800
+++ Tc =  30.1 min
+++ CA =  17.5
+++ Link #  7,  Flow depth =   1.800 m, Critical depth =   1.800 m
  PNC 7 8 0 180 2
  PFS 0 60 10 10 1 3 2 30 0.05
  PHJ 2 3 6
  PFP 3 7
  END
 END OF INPUT DATA.
```

****** HYDRA ******* (Version 6.1) ***** Date 07-16-1998
Page No 4

Example Two: Rational Method Analysis with Pressure Flow

*** MAIN STREET Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	-Solutions- Diam (mm)
1	137	450	57.570	.003	1.410	.923	1.147	.182	110	.017	279
			57.110		1.410	.923	1.038	.165			
2	229	750	56.590	.009	1.930	1.117	2.946	1.302	120	.219	432
			54.430		1.740	.928	2.450	1.082			
3	152	1350	53.770	.001	2.400	.937	3.214	4.600	275	2.925	1778
			53.620		2.580	1.118	1.170	1.675			

Length = 518. m Total length = 518. m

*** PINE STREET Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	-Solutions- Diam (mm)
4	107	375	59.630	.043	1.330	.924	3.387	.374	103	.011	279
			55.060		1.320	.914	3.286	.363			
5	168	1200	54.220	.001	2.160	.860	1.250	1.205	97		
			54.050		2.150	.850	1.098	1.242			
6	107	1350	53.770	.001	2.400	.937	1.365	1.954	114	.240	660
			53.660		2.540	1.078	1.198	1.714			
7	122	1800	53.310	.001	2.890	.940	2.670	6.796	181	3.042	1778
			53.180		2.960	1.010	1.475	3.754			

Length = 503. m Total length = 1021. m

Example Two: Rational Method Analysis with Pressure Flow

Pressurized Flow Simulations

```
Total Simulation time:           60 min
Incremental time:                10 min
Length of integration step:      10 s
Initial time:                   .00 min
Surcharge variables: itMax       30 iterations
                     surTol    .050

Printed output at the following  3 junctions

      2      3      6

          and for the following  2 pipes

      3      7
```

Pipe Characteristics

Pipe Number	Length (m)	Area (m^2)	Manning "n"	Max. Width (m)	Depth (m)	Junctions at Ends		Invert Height above Junctions	
1	137.160	.164	.013	.457	.457	1	2	.000	.520
2	228.600	.456	.013	.762	.762	2	3	.000	.660
3	152.400	1.423	.013	1.346	1.346	3	7	.000	.310
4	106.680	.114	.013	.381	.381	4	5	.000	.840
5	167.640	1.119	.013	1.194	1.194	5	6	.000	.280
6	106.680	1.423	.013	1.346	1.346	6	7	.000	.350
7	121.920	2.554	.013	1.803	1.803	7	8	.000	.000

Junction Characteristics

Junction Number	Ground Elev.	Crown Elev.	Invert Elev.	Connecting Pipes	
1	58.980	58.027	57.570	1	
2	58.520	57.567	56.590	1	2
3	56.170	55.192	53.770	2	3
4	60.960	60.011	59.630	4	
5	56.380	55.441	54.220	4	5

Example Two: Rational Method Analysis with Pressure Flow

Junction Characteristics

Junction Number	Ground Elev.	Crown Elev.	Invert Elev.	Connecting Pipes		
6	56.170	55.244	53.770	5	6	
7	56.200	55.113	53.310	3	6	7
8	56.140	54.983	53.180	7		

+++ Outfall control water-surface elev = 53.350 m

Summary of initial heads, flows and velocities

Initial heads, flows and velocities are zero

Junction Hydrographs Obtained by Simplified Rational Formula

Junction Number	Triangle Hydrograph Time (min)/Inflow (m^3/s)					
1	.00/	.000	21.00/	.182	42.00/	.000
2	.00/	.000	28.00/	1.134	56.00/	.000
3	.00/	.000	15.00/	3.949	30.00/	.000
4	.00/	.000	15.00/	.374	30.00/	.000
5	.00/	.000	15.00/	.838	30.00/	.000
6	.00/	.000	15.00/	.804	30.00/	.000
7	.00/	.000	15.00/	.678	30.00/	.000

*** WARNING: iCyc = 113 zero surface area computed at junction 2
... Check input data for high pipe.

Time History of Hydraulic Gradeline

	Junction 2		Junction 3		Junction 6	
	Grnd 58.520		Grnd 56.170		Grnd 56.170	
Time (h:min)	Elev (m)	Depth (m)	Elev (m)	Depth (m)	Elev (m)	Depth (m)
0:10	56.934	.344	55.702	1.932	55.624	1.854

Example Two: Rational Method Analysis with Pressure Flow

0:20	57.822	1.232	56.170	2.400	55.832	2.062
0:30	57.563	.973	54.841	1.071	54.740	.970
0:40	57.024	.434	54.402	.632	54.165	.395
0:50	56.842	.252	54.195	.425	53.895	.125
1: 0	56.620	.030	53.898	.128	53.804	.034

Summary of Junction Results

Junction Number	Ground /Invert Elev. (m)	Uppermost Pipe crown Elev. (m)	Maximum Computed Water Surface Elev	Time of Occurrence (h:min)	Surcharge at Max. Depth	Length of Surcharge (min)
1	58.980 57.570	58.027	58.980	0 : 19	.953	8.333
2	58.520 56.590	57.567	58.520	0 : 19	.953	11.000
3	56.170 53.770	55.192	56.170	0 : 11	.978	18.833
4	60.960 59.630	60.011	60.642	0 : 15	.631	1.000
5	56.380 54.220	55.441	56.380	0 : 10	.939	11.667
6	56.170 53.770	55.244	56.170	0 : 11	.926	14.333
7	56.200 53.310	55.113	56.200	0 : 11	1.087	14.833
8	56.140 53.180	54.983	53.180	0 : 0	.000	.000

Example Two: Rational Method Analysis with Pressure Flow

Time History of Flow and Velocity

Time (h:min)	Pipe 3 Flow m^3/s	Pipe 3 Vel m/s	Pipe 7 Flow m^3/s	Pipe 7 Vel m/s
0:10	3.130	2.062	2.034	3.028
0:20	2.409	1.620	2.259	2.159
0:30	1.382	1.137	2.228	2.244
0:40	.741	1.245	.855	1.627
0:50	.309	1.032	.373	1.243
1: 0	.021	.434	.049	.625

Summary of Pipe Results

Pipe Number	Design Flo/Vel m^3/s|m/s	Pipe Vertical Depth (mm)	Maximum Computed Flo/Vel m^3/s|m/s	Time of Occurrence (h:min)	Ratio of Max. to Design Flow	Max. Depth above Invert Pipe Ends Up (m)	Dn (m)
1	.172 1.049	458.	.182 1.239	0 : 21	1.1	1.410	1.410
2	1.129 2.476	763.	1.251 2.781	0 : 28	1.1	1.930	1.740
3	1.662 1.168	1347.	3.464 2.433	0 : 10	2.1	2.400	2.580
4	.379 3.321	382.	.378 3.313	0 : 15	1.0	1.012	1.320
5	1.225 1.094	1194.	1.202 1.240	0 : 15	1.0	2.160	2.120
6	1.701 1.195	1347.	2.554 1.794	0 : 18	1.5	2.400	2.540
7	3.773 1.477	1804.	4.836 3.028	0 : 11	1.3	2.890	.000

NORMAL END OF HYDRA

Example Three: Hydrographic Simulation and Design

Problem:

The site shown in figure 15 is a roadway section between two high points in the centerline profile. Six inlets are used to capture flow from eight identical pavement sections, each 0.20 ha in size. Gutter flow between the inlets is expected in situations where the inlet is not able to accept portions of the peak flow. Once captured by one of the inlets, the flow goes through a feeder network of pipes to the main outfall pipe which, after passing through a pond, empties into a downstream channel.

The hydrographic commands featured in this example are **STE** (step), **UHY** (user-hydrograph), **GUT** (gutter), and **INL** (inlet). The **STE** command sets the time step for the analysis. In this example, it is set at 2 min which is relatively small for many analyses. The **UHY** command enables the user to employ either a user-defined hydrograph or a HYDRO-generated hydrograph (whose file name is characterized by a QT extension). The **GUT** command allows for surface flow at the curb's edge. Finally, the **INL** command serves to transfer surface flow from a gutter into a pipe network below. Various inlet types can be analyzed or designed, either on grade or in a sump condition. The **SDI** (stage-discharge), **SST** (stage-storage), and **RES** (reservoir) commands route in-system flow through a reservoir.

Another transport command called **CHA** (channel) is introduced in this example. The **CHA** command models open channel flow for a variety of cross sections. The **CHA** and **GUT** commands, in addition to the **PIP** command, give the user significant flexibility in transporting flow.

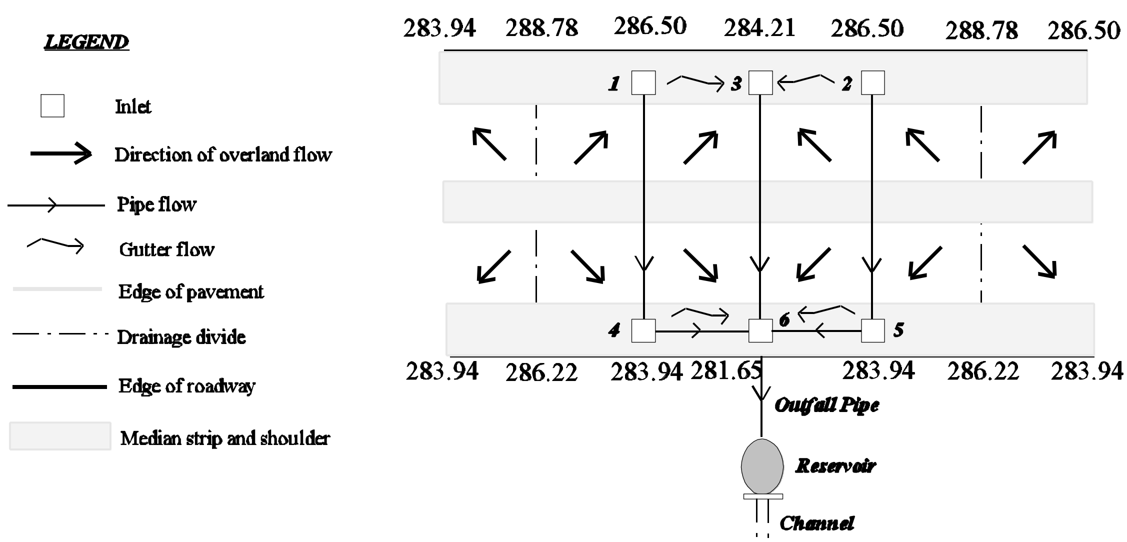

Figure 15. Hydrographic analysis.

Input File: HYDRA3.HDA

```
JOB Example Three:  Hydrographic Simulation and Design
SWI 3
CRI 1
PDA 0.013  300  1.52  1.22  0.762  0.001
PCO  300,150.9  375,167.3  450,187.0   525,206.7   600,226.4   675,242.8
     750,262.5  825,282.2  900,318.2  1050,357.6  1200,413.4  1350,469.2
    1500,525.0 1650,580.7 1800,675.9  2100,974.4
CST 1.5  0.457  0  0  0  0.152  8.12  1  0  1.99  1.05  0.78  2.04  1.99  0
EXC 1.52  0.94  7.62  1.48
TSL 0  0.25  3.05  0.25
STE 2
NEW PATH 1-4-6
UHY RUNOFF.QT
GUT 152.39  288.78  286.50  0.013  30  0.610  20
INL 1  1  1  101  0.610  1.22
PIP 26.82  286.50  283.94
UHY RUNOFF.QT
GUT 152.39  286.22  283.94  0.013  30  0.610  20
INL 4  1  1  104  0.610  1.22
PIP 152.39  283.94 281.65
HOL 4
NEW PATH 2-5-6
UHY RUNOFF.QT
GUT 152.39  288.78  286.50  0.013  30  0.610  20
INL 2  1  1  102  0.610  1.22
PIP 26.82  286.50  283.94
UHY RUNOFF.QT
GUT 152.39  286.22  283.94  0.013  30  0.610  20
INL 5  1  1  105  0.610  1.22
PIP 152.39  283.94  281.65
HOL 5
NEW PATH 3-6-OUTFALL
GET 101
UHY RUNOFF.QT
GUT 152.39  286.50  284.21  0.013  30  0.610  20
PUT 103
GET 102
UHY RUNOFF.QT
GUT 152.30  286.50  284.21  0.013  30  0.610  20
GET 103
INL 3  4  1  0  0.610  1.22
PIP 26.82  284.21  281.65
REC 4
REC 5
GET 104
UHY RUNOFF.QT
GUT 152.39  283.94  281.65  0.013  30  0.610  20
PUT 106
GET 105
UHY RUNOFF.QT
GUT 152.39  283.94  281.65  0.013  30  0.610  20
GET 106
INL 6  4  1  0  0.610  1.22
PIP 15.24  281.65  280.40  0  0  0  0  1  1
SDI 278.57,0  279.18,0.057  279.79,0.226  280.40,0.510  281.01,0.906
    281.62,1.42
SST 278.57,0  279.18,17.10  279.79,68.40  280.40,153.91  281.01,273.61
    281.62,427.52
RES 0  0
REM OUTFALL CHANNEL
CHA 304.78  278.57  277.96  0.034  1  1.22  1
END
```

Discussion of output:

The output illustrates that HYDRA provides warning and announcement messages as each command is processed. These messages can be helpful for debugging the system as well as providing design data. As exhibited in the output reservoir routing on in-system flow yields a maximum of 120 m^3 volume and maximum stage of 280.159 m.

Output file: HYDRA3.LST

```
        ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                              Page No   1
   Example Three:  Hydrographic Simulation and Design

 +++ Commands Read From File C:\HYDRA\HYDRA3.HDA
   JOB
   SWI 3
   CRI 1
   PDA 0.013  300  1.52  1.22  0.762  0.001
   PCO  300,150.9  375,167.3  450,187.0   525,206.7   600,226.4   675,242.8
     750,262.5  825,282.2  900,318.2  1050,357.6  1200,413.4  1350,469.2
    1500,525.0 1650,580.7 1800,675.9  2100,974.4
   CST 1.5  0.457  0  0  0  0.152  8.12  1  0  1.99  1.05  0.78  2.04  1.99  0
   EXC 1.52  0.94  7.62  1.48
   TSL 0  0.25  3.05  0.25
   STE 2
 +++ Step reset from 15.0 minutes
   NEW PATH 1-4-6
   UHY RUNOFF.QT
 +++ Notice:  Intermediate file has data in SI units
   GUT 152.39  288.78  286.50  0.013  30  0.610  20
 +++ Depth =   .083 m, Vel =   1.276 m/s,  Width =   2.171 m
 +++ Discharge =     .104 m^3/s, Slope =   .015 m/m
   INL 1  1  1  101  0.610  1.22
 +++ Inlet capacity =      .076 m^3/s
   PIP 26.82  286.50  283.94
 +++ Top width of trench ---> UP  1.1 m
                        ---> DN  1.1 m
 +++ Link #  1,  Flow depth =    .105 m, Critical depth =    .215 m
   UHY RUNOFF.QT
 +++ Notice:  Intermediate file has data in SI units
   GUT 152.39  286.22  283.94  0.013  30  0.610  20
 +++ Depth =   .083 m, Vel =   1.276 m/s,  Width =   2.171 m
 +++ Discharge =     .104 m^3/s, Slope =   .015 m/m
   INL 4  1  1  104  0.610  1.22
 +++ Inlet capacity =      .076 m^3/s
   PIP 152.39  283.94 281.65
 *** WARNING: Crowns not matched because cover and depth criteria override.
 +++ Top width of trench ---> UP  1.2 m
                        ---> DN  1.2 m
 +++ Link #  2,  Flow depth =    .236 m, Critical depth =    .287 m
   HOL 4
   NEW PATH 2-5-6
   UHY RUNOFF.QT
 +++ Notice:  Intermediate file has data in SI units
   GUT 152.39  288.78  286.50  0.013  30  0.610  20
 +++ Depth =   .083 m, Vel =   1.276 m/s,  Width =   2.171 m
 +++ Discharge =     .104 m^3/s, Slope =   .015 m/m
```

```
  INL 2  1  1  102  0.610  1.22
+++ Inlet capacity =      .076 m^3/s
  PIP 26.82  286.50  283.94
+++ Top width of trench ---> UP  1.1 m
                        ---> DN  1.1 m
+++ Link #  3,  Flow depth =    .105 m, Critical depth =    .215 m
  UHY RUNOFF.QT
```

```
  Example Three:  Hydrographic Simulation and Design

+++ Notice:  Intermediate file has data in SI units
  GUT 152.39  286.22  283.94  0.013  30  0.610  20
+++ Depth =   .083 m, Vel =   1.276 m/s,  Width =   2.171 m
+++ Discharge =     .104 m^3/s, Slope =   .015 m/m
  INL 5  1  1  105  0.610  1.22
+++ Inlet capacity =      .076 m^3/s
  PIP 152.39  283.94  281.65
*** WARNING: Crowns not matched because cover and depth criteria override.
+++ Top width of trench ---> UP  1.2 m
                        ---> DN  1.2 m
+++ Link #  4,  Flow depth =    .236 m, Critical depth =    .287 m
  HOL 5
  NEW PATH 3-6-OUTFALL
  GET 101
  UHY RUNOFF.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 152.39  286.50  284.21  0.013  30  0.610  20
+++ Depth =   .082 m, Vel =   1.278 m/s,  Width =   2.169 m
+++ Discharge =     .104 m^3/s, Slope =   .015 m/m
  PUT 103
  GET 102
  UHY RUNOFF.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 152.30  286.50  284.21  0.013  30  0.610  20
+++ Depth =   .082 m, Vel =   1.278 m/s,  Width =   2.169 m
+++ Discharge =     .104 m^3/s, Slope =   .015 m/m
  GET 103
  INL 3  4  1  0  0.610  1.22
Default orif area =    .670 m^2, Perimeter ( 3 sides) =   2.440 m
+++ Maximum Storage =          69. m^3
+++ Ponding Time =           16.0 min
  PIP 26.82  284.21  281.65
+++ Top width of trench ---> UP  1.1 m
                        ---> DN  1.1 m
+++ Link #  5,  Flow depth =    .108 m, Critical depth =    .218 m
  REC 4
  REC 5
  GET 104
  UHY RUNOFF.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 152.39  283.94  281.65  0.013  30  0.610  20
+++ Depth =   .082 m, Vel =   1.278 m/s,  Width =   2.169 m
+++ Discharge =     .104 m^3/s, Slope =   .015 m/m
  PUT 106
  GET 105
  UHY RUNOFF.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 152.39  283.94  281.65  0.013  30  0.610  20
+++ Depth =   .082 m, Vel =   1.278 m/s,  Width =   2.169 m
+++ Discharge =     .104 m^3/s, Slope =   .015 m/m
  GET 106
  INL 6  4  1  0  0.610  1.22
Default orif area =    .670 m^2, Perimeter ( 3 sides) =   2.440 m
+++ Maximum Storage =          69. m^3
+++ Ponding Time =           16.0 min
```

```
   Example Three:  Hydrographic Simulation and Design

   PIP 15.24  281.65  280.40  0  0  0  0  1  1
 *** WARNING: Crowns not matched because cover and depth criteria override.
 +++ Top width of trench ---> UP  1.3 m
                        ---> DN  1.2 m

                              SYSTEM HYDROGRAPH in m^3/s
```

```
.460+)))))3)))*))*))*)))))3)))))))))))))))3)))))))))))))))3))))))))))))))),
 D        *     .            *  .               .               .
*
 i        *     *               .               .               .
*
 s    .4003     .               .               .               .
3
 c        *     .               *               .               .
*
 h        *     .               .               .               .
*
 a        *  *  .               .               .               .
*
 r    .3003     .               .  *            .               .
3
 g        *     .               .               .               .
*
 e        *     .               .               .               .
*
          *     .               .      *        .               .
*
 i    .2003     .               .               .               .
3
 n        *     .               .         *  *  *  *   *  *     .
*
          *     .               .               .               .
*
 m        *     .               .               .               .
*
 ^    .1003     .               .               .               .
3
 3        *     .               .               .               .
*
 /        *     .               .               .            *  .
*
 s        *     .               .               .               .
*

.000.)))))3)))))))))))))))3)))))))))))))))3)))))))))))))))3))*))*)))*))*))*
       6.    10.             20.             30.             40.
50.

                                     Time,  t (min)
```

Example Three: Hydrographic Simulation and Design

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
6.	.156	46.	.000	0.	.000	0.	.000	0.	.000
8.	.320	48.	.000	0.	.000	0.	.000	0.	.000
10.	.424	50.	.000	0.	.000	0.	.000	0.	.000
12.	.459	0.	.000	0.	.000	0.	.000	0.	.000
14.	.460	0.	.000	0.	.000	0.	.000	0.	.000
16.	.458	0.	.000	0.	.000	0.	.000	0.	.000
18.	.427	0.	.000	0.	.000	0.	.000	0.	.000
20.	.357	0.	.000	0.	.000	0.	.000	0.	.000
22.	.288	0.	.000	0.	.000	0.	.000	0.	.000
24.	.218	0.	.000	0.	.000	0.	.000	0.	.000
26.	.173	0.	.000	0.	.000	0.	.000	0.	.000
28.	.173	0.	.000	0.	.000	0.	.000	0.	.000
30.	.170	0.	.000	0.	.000	0.	.000	0.	.000
32.	.168	0.	.000	0.	.000	0.	.000	0.	.000
34.	.166	0.	.000	0.	.000	0.	.000	0.	.000
36.	.164	0.	.000	0.	.000	0.	.000	0.	.000
38.	.060	0.	.000	0.	.000	0.	.000	0.	.000
40.	.009	0.	.000	0.	.000	0.	.000	0.	.000
42.	.006	0.	.000	0.	.000	0.	.000	0.	.000
44.	.002	0.	.000	0.	.000	0.	.000	0.	.000

```
+++ System hydrograph registers full at time =    50. min
+++ Link #  6,  Flow depth =    .296 m, Critical depth =    .372 m
  SDI 278.57,0  279.18,0.057  279.79,0.226  280.40,0.510  281.01,0.906
   281.62,1.42
```

Example Three: Hydrographic Simulation and Design

STAGE - DISCHARGE CURVE

```
1.4+))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))*
 D        *         .           .           .           .           .
*
 i        *         .           .           .           .           .
*
 s        *         .           .           .           .           .
*
 c        *         .           .           .           .           .
*
 h     1.03         .           .           .           .           .
3
 a        *         .           .           .           .           .
*
 r        *         .           .           .           .         * .
*
 g        *         .           .           .           .           .
*
 e        *         .           .           .           .           .
*
          *         .           .           .           .           .
*
 i        *         .           .           .           .           .
*
 n      .53         .           .           .       *   .           .
3
          *         .           .           .           .           .
*
 m        *         .           .           .           .           .
*
 ^        *         .           .           .           .           .
*
 3        *         .           .     *     .           .           .
*
 /        *         .           .           .           .           .
*
 s        *         .   *       .           .           .           .
*

.0*))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))-
    278.6      279.0       279.5       280.0       280.5       281.0
281.6
```

Elevation, EL (m)

PLOT-DATA (Stage, EL(m) Vs. Discharge, Q(m^3/s))

EL	Q	EL	Q	EL	Q	EL	Q	EL	Q

```
 278.570    .000 281.010    .906    .000    .000    .000    .000    .000    .000
 279.180    .057 281.620   1.420    .000    .000    .000    .000    .000    .000
 279.790    .226    .000    .000    .000    .000    .000    .000    .000    .000
 280.400    .510    .000    .000    .000    .000    .000    .000    .000    .000
________________________________________________________________________________

   SST 278.57,0  279.18,17.10  279.79,68.40  280.40,153.91  281.01,273.61
    281.62,427.52
```

Example Three: Hydrographic Simulation and Design

STAGE - STORAGE CURVE

```
427.5+))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))*
         *         .           .           .           .           .
*
         *         .           .           .           .           .
*
 V       *         .           .           .           .           .
*
 o  300.03         .           .           .           .           .
3
 l       *         .           .           .           .           .
*
 u       *         .           .           .           .           .
*
 m       *         .           .           .           .         * .
*
 e       *         .           .           .           .           .
*
    200.03         .           .           .           .           .
3
 i       *         .           .           .           .           .
*
 n       *         .           .           .           .           .
*
         *         .           .           .       *   .           .
*
 m       *         .           .           .           .           .
*
 ^  100.03         .           .           .           .           .
3
 3       *         .           .           .           .           .
*
         *         .           .     *     .           .           .
*
         *         .           .           .           .           .
*
         *         .   *       .           .           .           .
*

.0*))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))-
     278.6      279.0       279.5       280.0       280.5       281.0
281.6
```

Elevation, EL (m)

PLOT-DATA (Stage, EL(m) vs. Storage, V(m^3))

EL	V	EL	V	EL	V	EL	V	EL	V

```
278.570     .000 281.010 273.610    .000    .000    .000    .000    .000    .000
279.180   17.100 281.620 427.520    .000    .000    .000    .000    .000    .000
279.790   68.400    .000    .000    .000    .000    .000    .000    .000    .000
280.400  153.910    .000    .000    .000    .000    .000    .000    .000    .000
```

```
  RES 0  0
+++ Max. volume =     120. m^3 at an elevation of  280.159 m
  REM OUTFALL CHANNEL
  CHA 304.78  278.57  277.96  0.034  1  1.22  1
+++ Link   7, Max. shear stress =      8.3 Pa
  END
 END OF INPUT DATA.
```

Example Three: Hydrographic Simulation and Design

*** PATH 1-4-6 Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
1	27	300	284.955	.095	1.545	1.220	3.528	.076	4108.
			282.395		1.545		4.227	.299	
2	152	375	282.289	.015	1.651	1.220	2.099	.151	25933.
			280.024		1.626		1.935	.214	

Length = 179. m Total length = 179. m
Cost = 30041. Total Cost = 30041.

*** PATH 2-5-6 Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
3	27	300	284.955	.095	1.545	1.220	3.528	.076	4108.
			282.395		1.545		4.227	.299	
4	152	375	282.289	.015	1.651	1.220	2.099	.151	25933.
			280.024		1.626		1.935	.214	

Length = 179. m Total length = 179. m
Cost = 30041. Total Cost = 30041.

Example Three: Hydrographic Simulation and Design

*** PATH 3-6-OUTFALL   Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
5	27	300	282.665	.095	1.545	1.220	3.561	.079	4108.
			280.105		1.545		4.227	.299	
6	15	375	279.918	.075	1.732	1.220	4.950	.460	2595.
			278.774		1.626		4.349	.480	

Length =   42. m   Total length =   400. m
Cost   = 6703.   Total Cost   = 66785.

*** PATH 3-6-OUTFALL   Channel

Link	Length (m)	-Channel Shape-- Left	Ctr (m)	Right	Slope (m/m)	Invert Up/Dn (m)	Surface Up/Dn (m)	Depth (m)	Surf Width (m)	Flow (m^3/s)	Vel (m/s)
7	305	1.0	1.2	1.0	.002	278.570	278.994	.424	2.067	.398	.574
						277.960	278.384				

Length =   347. m   Total length =   705. m

```
          ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                               Page No   9

   Example Three:  Hydrographic Simulation and Design


 NORMAL END OF HYDRA
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

Problem:

The example shown in figure 16 demonstrates the application of HYDRA to an existing storm sewer system by implementing hydrographic flow by way of the **UHY** command. The hydrographs appear in tabular form with a fixed format in a file with a **QT** extension. In addition, the time step on the **STE** command reflects the uniform time interval that occurs in the tabular user hydrographs. User-defined inlets are utilized and have sufficient efficiencies to capture the flow specified in the **QT** files. The tailwater elevation is given in the **TWE** command to be the invert of the outlet pipe. This elevation is necessary for the hydraulic gradeline and pressurized flow computations.

The pressure flow simulation command, **PFS**, establishes the control parameters for the pressurized flow computations. In this example, the simulation time is set for a duration in which the maximum surcharges within the system should occur (based on the times to peak of inflow hydrographs). The parameters, **ITMAX** and **SURTOL**, are given acceptable values of 30 and 0.05, respectively, to ensure adequate accuracy.

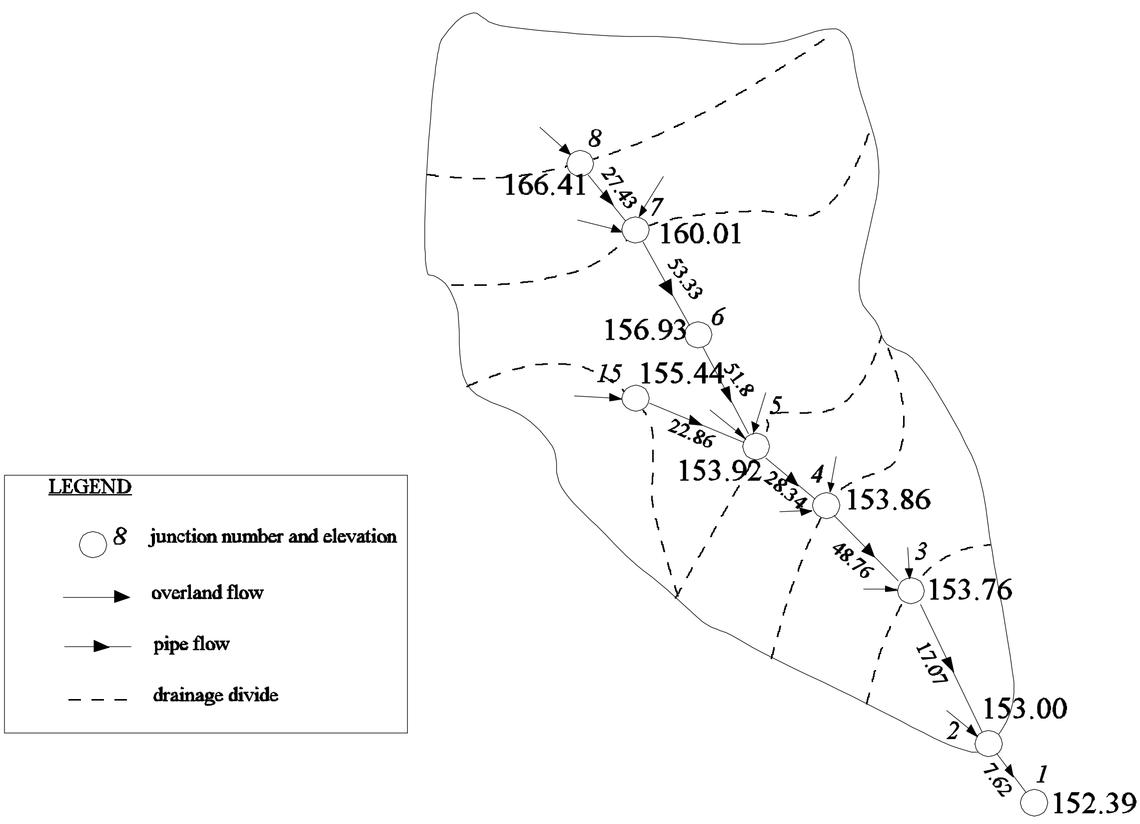

Figure 16. Hydrographic analysis with pressure flow.

Data file: HYDR15.QT

```
Example 4 - hydrograph for junctions 8 & 15
   27
         1        0.0    0.0000
         2       3.00    0.0011
         3       6.00    0.0059
         4       9.00    0.0127
         5      12.00    0.0224
         6      15.00    0.0345
         7      18.00    0.0481
         8      21.00    0.0617
         9      24.00    0.0682
        10      27.00    0.0753
        11      30.00    0.0793
        12      33.00    0.0801
        13      36.00    0.0790
        14      39.00    0.0750
        15      42.00    0.0702
        16      45.00    0.0643
        17      48.00    0.0566
        18      51.00    0.0490
        19      54.00    0.0422
        20      57.00    0.0374
        21      60.00    0.0326
        22      63.00    0.0283
        23      66.00    0.0246
        24      69.00    0.0218
        25      72.00    0.0193
        26      75.00    0.0170
        27      78.00    0.0144
```

Data file: HYDR4.QT

```
Example 4 - hydrograph for junctions 4 & 7
   26
         1        0.0    0.0000
         2       3.00    0.0017
         3       6.00    0.0079
         4       9.00    0.0170
         5      12.00    0.0294
         6      15.00    0.0453
         7      18.00    0.0631
         8      21.00    0.0813
         9      24.00    0.0937
        10      27.00    0.1022
        11      30.00    0.1053
        12      33.00    0.1033
        13      36.00    0.0968
        14      39.00    0.0886
        15      42.00    0.0790
        16      45.00    0.0696
        17      48.00    0.0589
        18      51.00    0.0515
        19      54.00    0.0442
```

```
        20      57.00    0.0391
        21      60.00    0.0337
        22      63.00    0.0294
        23      66.00    0.0252
        24      69.00    0.0221
        25      72.00    0.0190
        26      75.00    0.0164
```

Data file: HYDR05.QT

```
Example 4 - hydrograph for junction 5
   27
         1        0.0    0.0000
         2       3.00    0.0045
         3       6.00    0.0227
         4       9.00    0.0439
         5      12.00    0.0770
         6      15.00    0.1220
         7      18.00    0.1815
         8      21.00    0.2330
         9      24.00    0.2693
        10      27.00    0.2865
        11      30.00    0.2995
        12      33.00    0.3027
        13      36.00    0.2987
        14      39.00    0.2837
        15      42.00    0.2605
        16      45.00    0.2387
        17      48.00    0.2143
        18      51.00    0.1849
        19      54.00    0.1583
        20      57.00    0.1365
        21      60.00    0.1195
        22      63.00    0.1059
        23      66.00    0.0929
        24      69.00    0.0804
        25      72.00    0.0691
        26      75.00    0.0597
        27      78.00    0.0515
```

Data file: HYDR02.QT

```
Example 4 - hydrograph for junction 2
   26
         1        0.0    0.0000
         2       3.00    0.0003
         3       6.00    0.0008
         4       9.00    0.0020
         5      12.00    0.0031
         6      15.00    0.0048
         7      18.00    0.0068
         8      21.00    0.0091
         9      24.00    0.0102
        10      27.00    0.0113
        11      30.00    0.0116
        12      33.00    0.0113
```

```
        13     36.00    0.0108
        14     39.00    0.0096
        15     42.00    0.0088
        16     45.00    0.0079
        17     48.00    0.0065
        18     51.00    0.0057
        19     54.00    0.0048
        20     57.00    0.0042
        21     60.00    0.0037
        22     63.00    0.0034
        23     66.00    0.0028
        24     69.00    0.0023
        25     72.00    0.0020
        26     75.00    0.0017
```

Input file: HYDRA4.HDA

```
JOB Example Four: Hydrographic Analysis and Pressure Flow Simulation
SWI 3
CRI 0
STE 3.0
TWE 150.81
HGL 1
NEW LINK 8 TO 5
REM LINE 8 TO 6
PDA 0.022  300  1.22  0.914  0.610  0.001
UHY HYDR15.QT
GUT 30.48  167.14  166.41  0.013  50  0.914  40  0  1
INL 8  7  0  101 0 0 0 0 0 0 1
EFF 0  100  2.83  100
PIP 27.43  166.41  160.01  165.16  158.76  -300
PNC 8  7  0.914  22.0  0
REM LINK 7 TO 6
PDA 0.014  300  1.22  0.914  0.610  0.001
UHY HYDR04.QT
GUT 45.72  160.62  160.01  0.013  50  0.610  40  0  1
INL 7  7  0  102  0 0 0 0 0 0 1
EFF 0  100  2.83  100
PIP 53.33  160.01  156.93  158.58  155.50  -450
PNC 7  6  0.914  0.0  0
REM LINK 6 TO 5
PIP 51.81  156.93  153.92  155.50  152.51  -450
PNC 6  5  0.914  40.0  0
HOL 5
NEW MAINLINE 15 TO 1
REM LINK 15 TO 5
UHY HYDR15.QT
GUT 30.48  156.05  155.44  0.013  50  0.914  40  0  1
INL 15  7  0  103  0 0 0 0 0 0 1
EFF 0 100 2.83 100
PIP 22.86  155.44  153.92  154.19  152.67  -300
PNC 15  5  1.22  0  0
REM LINK 5 TO 4
UHY HYDR05.QT
GUT 15.24  154.07  153.92  0.013  50  1.52  30  0  1
INL 5  7  0  104  0 0 0 0 0 0 1
EFF 0  100  2.83  100
REC 5
PIP 28.34  153.92  153.61  152.48  152.21  -450
PNC 5  4  1.22  11.0  0
REM LINK 4 TO 3
UHY HYDR04.QT
GUT 36.57  154.53  153.86  0.013  30  1.52  30  0  1
INL 4  7  0  105  0 0 0 0 0 0 1
EFF 0  100  2.83  100
```

```
PIP 48.76  153.86  153.76  151.90  151.84  -750
PNC 4  3  1.22  0.0  0
REM LINK 3 TO 2
UHY HYDR04.QT
GUT 54.86  154.62  153.76  0.013  50  0.914  40  0  1
INL 6  7  0  106
EFF 0  100  2.83  100
PIP 28.95  153.76  153.00  151.84  151.26  -750
PNC 3  2  1.22  48  0
REM LINK 2 TO 1
UHY HYDR02.QT
GUT 22.86  153.46  153.00  0.013  50   0.914  40  0  1
INL 7  7  0  107  0 0 0 0 0 0 1
EFF 0  100  2.83  100
PIP 7.62  153.00  152.39  151.11  150.81  -600
PNC 2  1  0  0.0  2
PFS 0  60  3.0  5  0  9  8  30  0.05
PHJ 8  7  6  15  5  4  3  2  1
PFP 1  2   3  4  5  6   7  8
IQV 2.0  5.1  0  0  0  0  2.5  6.4
IDY IDY 0.0  0.0  0.0  0.0  0.6  0.0  0.0  0.0
END
```

Discussion of output:

The hydraulic gradeline (HGL) and pressure flow simulation (PFS) options present surcharge conditions of the storm sewer system. Generally, the HGL results are more conservative than the PFS results which the output depicts. If the detailed output had been requested, some computed depths and flows may have been followed by an asterisk. An asterisk following a depth indicates surcharging while an asterisk following a flow value indicates that the flow was set to normal flow. The PFS algorithm uses a convention in which the outfall is a pipe as opposed to a node. For this reason, it generates a synthetic outfall denoted as pipe 90009.

Only the first 1,000 lines of the output file will be displayed in the HYDRAIN editor. The entire file may be viewed in another text editor. See page 34 in Volume I: HYDRAIN for more information.

Output file: HYDRA4.LST

```
          ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                               Page No   1

   Example Four: Hydrographic Analysis and Pressure Flow Simulation

 +++ Commands Read From File C:\HYDRA\HYDRA4.HDA
   JOB
   SWI 3
   CRI 0
   STE 3.0
 +++ Step reset from 15.0 minutes
   TWE 150.81
   HGL 1
   NEW LINK 8 TO 5
   REM LINE 8 TO 6
   PDA 0.022  300  1.22  0.914  0.610  0.001
   UHY HYDR15.QT
 +++ Notice:  Intermediate file has data in SI units
   GUT 30.48  167.14  166.41  0.013  50  0.914  40  0  1
 +++ Depth =    .054 m, Vel =   1.246 m/s,  Width =   2.493 m
 +++ Discharge =     .080 m^3/s, Slope =   .024 m/m

                              GUTTER HYDROGRAPH in m^3/s


.080+))))))**)))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3))))))))),
 D       *     *  *  .           .           .           .           .
*
 i       *         * .           .           .           .           .
*
 s       *    *      .           .           .           .           .
*
 c   .0603   *     * .           .           .           .           .
3
 h       *           .           .           .           .           .
*
 a       *          *.           .           .           .           .
*
 r       *            *          .           .           .           .
*
 g       *   *       .           .           .           .           .
*
 e   .0403           .*          .           .           .           .
3
         *           .*          .           .           .           .
*
 i       *  *        . *         .           .           .           .
*
 n       *           .  *        .           .           .           .
*
         *           .  *        .           .           .           .
*
 m   .0203 *         .   **      .           .           .           .
3
 ^       *           .     *     .           .           .           .
*
```

```
 3       **          .     *     .          .          .          .
*
 /       *           .           .          .          .          .
*
 s       **          .           .          .          .          .
*

.000*))))))))))))3))))))*****3**********3**********3**********3**********
      3.         50.        100.        150.       200.        250.
288.

                                  Time, t (min)
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
3.	.001	63.	.028	123.	.000	183.	.000	243.	.000
6.	.006	66.	.025	126.	.000	186.	.000	246.	.000
9.	.013	69.	.022	129.	.000	189.	.000	249.	.000
12.	.022	72.	.019	132.	.000	192.	.000	252.	.000
15.	.034	75.	.017	135.	.000	195.	.000	255.	.000
18.	.048	78.	.014	138.	.000	198.	.000	258.	.000
21.	.062	81.	.000	141.	.000	201.	.000	261.	.000
24.	.068	84.	.000	144.	.000	204.	.000	264.	.000
27.	.075	87.	.000	147.	.000	207.	.000	267.	.000
30.	.079	90.	.000	150.	.000	210.	.000	270.	.000
33.	.080	93.	.000	153.	.000	213.	.000	273.	.000
36.	.079	96.	.000	156.	.000	216.	.000	276.	.000
39.	.075	99.	.000	159.	.000	219.	.000	279.	.000
42.	.070	102.	.000	162.	.000	222.	.000	282.	.000
45.	.064	105.	.000	165.	.000	225.	.000	285.	.000
48.	.057	108.	.000	168.	.000	228.	.000	288.	.000
51.	.049	111.	.000	171.	.000	231.	.000	0.	.000
54.	.042	114.	.000	174.	.000	234.	.000	0.	.000
57.	.037	117.	.000	177.	.000	237.	.000	0.	.000
60.	.033	120.	.000	180.	.000	240.	.000	0.	.000

```
+++ System hydrograph registers full at time =   288. min
  INL 8  7  0  101 0 0 0 0 0 0 1
  EFF 0  100  2.83  100
  PIP 27.43  166.41  160.01  165.16  158.76  -300
+++ Link #  1,  Flow depth =    .111 m, Critical depth =    .220 m
  PNC 8  7  0.914  22.0  0
  REM LINK 7 TO 6
  PDA 0.014  300  1.22  0.914  0.610  0.001
  UHY HYDR04.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 45.72  160.62  160.01  0.013  50  0.610  40  0  1
+++ Depth =   .066 m, Vel =   1.059 m/s,  Width =   3.139 m
+++ Discharge =     .105 m^3/s, Slope =   .013 m/m
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

GUTTER HYDROGRAPH in m^3/s

```
.105+))))))*))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3))))))))),
 D        *      *      .           .           .           .           .
*
 i        *     *  *    .           .           .           .           .
*
 s        *         *   .           .           .           .           .
*
 c   .0753    *         .           .           .           .           .
3
 h        *          *  .           .           .           .           .
*
 a        *          *  .           .           .           .           .
*
 r        *             .           .           .           .           .
*
 g        *   *       *.            .           .           .           .
*
 e   .0503              .           .           .           .           .
3
          *             *           .           .           .           .
*
 i        *  *          .*          .           .           .           .
*
 n        *             .*          .           .           .           .
*
          *             .  *        .           .           .           .
*
 m   .0253 *            .   *       .           .           .           .
3
 ^        *             .    *      .           .           .           .
*
 3        **            .     **    .           .           .           .
*
 /        *             .           .           .           .           .
*
 s        **            .           .           .           .           .
*

.000*)))))))))))3)))))******3**********3**********3**********3**********
      4.          50.        100.        150.        200.        250.
289.
```

Time, t (min)

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
4.	.002	64.	.029	124.	.000	184.	.000	244.	.000
7.	.008	67.	.025	127.	.000	187.	.000	247.	.000
10.	.017	70.	.022	130.	.000	190.	.000	250.	.000
13.	.029	73.	.019	133.	.000	193.	.000	253.	.000
16.	.045	76.	.016	136.	.000	196.	.000	256.	.000
19.	.063	79.	.000	139.	.000	199.	.000	259.	.000
22.	.081	82.	.000	142.	.000	202.	.000	262.	.000
25.	.094	85.	.000	145.	.000	205.	.000	265.	.000
28.	.102	88.	.000	148.	.000	208.	.000	268.	.000
31.	.105	91.	.000	151.	.000	211.	.000	271.	.000
34.	.103	94.	.000	154.	.000	214.	.000	274.	.000
37.	.097	97.	.000	157.	.000	217.	.000	277.	.000
40.	.089	100.	.000	160.	.000	220.	.000	280.	.000
43.	.079	103.	.000	163.	.000	223.	.000	283.	.000
46.	.070	106.	.000	166.	.000	226.	.000	286.	.000
49.	.059	109.	.000	169.	.000	229.	.000	289.	.000
52.	.052	112.	.000	172.	.000	232.	.000	0.	.000
55.	.044	115.	.000	175.	.000	235.	.000	0.	.000
58.	.039	118.	.000	178.	.000	238.	.000	0.	.000
61.	.034	121.	.000	181.	.000	241.	.000	0.	.000

```
+++ System hydrograph registers full at time =   289. min
  INL 7  7  0  102  0 0 0 0 0 0 1
  EFF 0  100  2.83  100
  PIP 53.33  160.01  156.93  158.58  155.50  -450
+++ Link #  2,  Flow depth =    .167 m, Critical depth =    .302 m
  PNC 7  6  0.914  0.0  0
  REM LINK 6 TO 5
  PIP 51.81  156.93  153.92  155.50  152.51  -450
+++ Link #  3,  Flow depth =    .167 m, Critical depth =    .302 m
  PNC 6  5  0.914  40.0  0
  HOL 5
  NEW MAINLINE 15 TO 1
  REM LINK 15 TO 5
  UHY HYDR15.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 30.48  156.05  155.44  0.013  50  0.914  40  0  1
+++ Depth =   .056 m, Vel =   1.166 m/s,  Width =   2.581 m
+++ Discharge =     .080 m^3/s, Slope =   .020 m/m
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

GUTTER HYDROGRAPH in m^3/s

```
.080+))))))**)))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3))))))))),
 D        *      *   *   .            .            .            .            .
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 i        *          *  .            .            .            .            .
*
 s        *     *       .            .            .            .            .
*
 c    .0603    *      *  .            .            .            .            .
3
 h        *             .            .            .            .            .
*
 a        *           *.            .            .            .            .
*
 r        *             *            .            .            .            .
*
 g        *    *        .            .            .            .            .
*
 e    .0403             .*           .            .            .            .
3
          *             .*           .            .            .            .
*
 i        *  *          .  *         .            .            .            .
*
 n        *             .   *        .            .            .            .
*
          *             .   *        .            .            .            .
*
 m    .0203 *           .     **     .            .            .            .
3
 ^        *             .      *     .            .            .            .
*
 3        **            .      *     .            .            .            .
*
 /        *             .            .            .            .            .
*
 s        **            .            .            .            .            .
*

.000*)))))))))))3))))))*****3**********3**********3**********3**********
       3.          50.        100.        150.        200.        250.
288.
```

Time, t (min)

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
3.	.001	63.	.028	123.	.000	183.	.000	243.	.000
6.	.006	66.	.025	126.	.000	186.	.000	246.	.000
9.	.013	69.	.022	129.	.000	189.	.000	249.	.000
12.	.022	72.	.019	132.	.000	192.	.000	252.	.000
15.	.034	75.	.017	135.	.000	195.	.000	255.	.000
18.	.048	78.	.014	138.	.000	198.	.000	258.	.000
21.	.062	81.	.000	141.	.000	201.	.000	261.	.000
24.	.068	84.	.000	144.	.000	204.	.000	264.	.000
27.	.075	87.	.000	147.	.000	207.	.000	267.	.000
30.	.079	90.	.000	150.	.000	210.	.000	270.	.000
33.	.080	93.	.000	153.	.000	213.	.000	273.	.000
36.	.079	96.	.000	156.	.000	216.	.000	276.	.000
39.	.075	99.	.000	159.	.000	219.	.000	279.	.000
42.	.070	102.	.000	162.	.000	222.	.000	282.	.000
45.	.064	105.	.000	165.	.000	225.	.000	285.	.000
48.	.057	108.	.000	168.	.000	228.	.000	288.	.000
51.	.049	111.	.000	171.	.000	231.	.000	0.	.000
54.	.042	114.	.000	174.	.000	234.	.000	0.	.000
57.	.037	117.	.000	177.	.000	237.	.000	0.	.000
60.	.033	120.	.000	180.	.000	240.	.000	0.	.000

```
+++ System hydrograph registers full at time =   288. min
  INL 15  7  0  103  0 0 0 0 0 0 1
  EFF 0 100 2.83 100
  PIP 22.86  155.44  153.92  154.19  152.67  -300
+++ Link #  4,  Flow depth =    .123 m, Critical depth =    .220 m
  PNC 15  5  1.22  0  0
  REM LINK 5 TO 4
  UHY HYDR05.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 15.24  154.07  153.92  0.013  50  1.52  30  0  1
+++ Depth =   .115 m, Vel =   1.271 m/s,  Width =   4.719 m
+++ Discharge =     .303 m^3/s, Slope =   .010 m/m
```

```
         ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                               Page No   7

   Example Four: Hydrographic Analysis and Pressure Flow Simulation

                             GUTTER HYDROGRAPH in m^3/s


.303+))))))**)))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3))))))))),
 D       *     *  *  .           .           .           .           .
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 i       *    *      .           .           .           .           .
*
 s       *         * .           .           .           .           .
*
 c       *   *     * .           .           .           .           .
*
 h   .2003           .           .           .           .           .
3
 a       *          *.           .           .           .           .
*
 r       *           *           .           .           .           .
*
 g       *   *       .           .           .           .           .
*
 e       *           .*          .           .           .           .
*
         *           .*          .           .           .           .
*
 i       *  *        .  *        .           .           .           .
*
 n   .1003           .   *       .           .           .           .
3
         *           .   *       .           .           .           .
*
 m       * *         .    *      .           .           .           .
*
 ^       *           .     **    .           .           .           .
*
 3       **          .      *    .           .           .           .
*
 /       *           .           .           .           .           .
*
 s       **          .           .           .           .           .
*

.000*)))))))))))3))))))*****3**********3**********3**********3*********
      3.          50.        100.        150.        200.        250.
288.

                                            Time, t (min)
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
3.	.004	63.	.106	123.	.000	183.	.000	243.	.000
6.	.023	66.	.093	126.	.000	186.	.000	246.	.000
9.	.044	69.	.080	129.	.000	189.	.000	249.	.000
12.	.077	72.	.069	132.	.000	192.	.000	252.	.000
15.	.122	75.	.060	135.	.000	195.	.000	255.	.000
18.	.182	78.	.052	138.	.000	198.	.000	258.	.000
21.	.233	81.	.000	141.	.000	201.	.000	261.	.000
24.	.269	84.	.000	144.	.000	204.	.000	264.	.000
27.	.287	87.	.000	147.	.000	207.	.000	267.	.000
30.	.299	90.	.000	150.	.000	210.	.000	270.	.000
33.	.303	93.	.000	153.	.000	213.	.000	273.	.000
36.	.299	96.	.000	156.	.000	216.	.000	276.	.000
39.	.284	99.	.000	159.	.000	219.	.000	279.	.000
42.	.261	102.	.000	162.	.000	222.	.000	282.	.000
45.	.239	105.	.000	165.	.000	225.	.000	285.	.000
48.	.214	108.	.000	168.	.000	228.	.000	288.	.000
51.	.185	111.	.000	171.	.000	231.	.000	0.	.000
54.	.158	114.	.000	174.	.000	234.	.000	0.	.000
57.	.137	117.	.000	177.	.000	237.	.000	0.	.000
60.	.119	120.	.000	180.	.000	240.	.000	0.	.000

```
+++ System hydrograph registers full at time =   288. min
  INL 5  7  0  104  0 0 0 0 0 0 1
  EFF 0  100  2.83  100
  REC 5
  PIP 28.34  153.92  153.61  152.48  152.21  -450
+++ Link #  5,  Flow depth =    .450 m, Critical depth =    .450 m
  PNC 5  4  1.22  11.0  0
  REM LINK 4 TO 3
  UHY HYDR04.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 36.57  154.53  153.86  0.013  30  1.52  30  0  1
+++ Depth =   .072 m, Vel =   1.357 m/s,  Width =   2.157 m
+++ Discharge =     .105 m^3/s, Slope =   .018 m/m
```

```
         ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                               Page No   9

   Example Four: Hydrographic Analysis and Pressure Flow Simulation

                                       GUTTER HYDROGRAPH in m^3/s

.105+))))))*))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3))))))))),
 D        *     *     .           .           .           .           .
*
 i        *    *  *   .           .           .           .           .
*
 s        *        *  .           .           .           .           .
*
 c    .0753   *       .           .           .           .           .
3
 h        *         * .           .           .           .           .
*
 a        *         * .           .           .           .           .
*
 r        *           .           .           .           .           .
*
 g        *   *      *.           .           .           .           .
*
 e    .0503           .           .           .           .           .
3
          *           *           .           .           .           .
*
 i        *  *        .*          .           .           .           .
*
 n        *           .*          .           .           .           .
*
          *           . *         .           .           .           .
*
 m    .0253 *         .  *        .           .           .           .
3
 ^        *           .   *       .           .           .           .
*
 3        **          .    **     .           .           .           .
*
 /        *           .           .           .           .           .
*
 s        **          .           .           .           .           .
*

.000*)))))))))))3)))))******3**********3**********3**********3**********
      3.         50.        100.        150.        200.        250.
288.

                                            Time, t (min)
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
3.	.002	63.	.029	123.	.000	183.	.000	243.	.000
6.	.008	66.	.025	126.	.000	186.	.000	246.	.000
9.	.017	69.	.022	129.	.000	189.	.000	249.	.000
12.	.029	72.	.019	132.	.000	192.	.000	252.	.000
15.	.045	75.	.016	135.	.000	195.	.000	255.	.000
18.	.063	78.	.000	138.	.000	198.	.000	258.	.000
21.	.081	81.	.000	141.	.000	201.	.000	261.	.000
24.	.094	84.	.000	144.	.000	204.	.000	264.	.000
27.	.102	87.	.000	147.	.000	207.	.000	267.	.000
30.	.105	90.	.000	150.	.000	210.	.000	270.	.000
33.	.103	93.	.000	153.	.000	213.	.000	273.	.000
36.	.097	96.	.000	156.	.000	216.	.000	276.	.000
39.	.089	99.	.000	159.	.000	219.	.000	279.	.000
42.	.079	102.	.000	162.	.000	222.	.000	282.	.000
45.	.070	105.	.000	165.	.000	225.	.000	285.	.000
48.	.059	108.	.000	168.	.000	228.	.000	288.	.000
51.	.052	111.	.000	171.	.000	231.	.000	0.	.000
54.	.044	114.	.000	174.	.000	234.	.000	0.	.000
57.	.039	117.	.000	177.	.000	237.	.000	0.	.000
60.	.034	120.	.000	180.	.000	240.	.000	0.	.000

```
+++ System hydrograph registers full at time =   288. min
  INL 4  7  0  105  0 0 0 0 0 0 1
  EFF 0  100  2.83  100
  PIP 48.76  153.86  153.76  151.90  151.84  -750
+++ Link #  6,  Flow depth =    .750 m, Critical depth =    .750 m
  PNC 4  3  1.22  0.0  0
  REM LINK 3 TO 2
  UHY HYDR04.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 54.86  154.62  153.76  0.013  50  0.914  40  0  1
+++ Depth =   .065 m, Vel =   1.133 m/s,  Width =   3.014 m
+++ Discharge =     .105 m^3/s, Slope =   .016 m/m
```

```
          ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                                Page No  11

   Example Four: Hydrographic Analysis and Pressure Flow Simulation

                              GUTTER HYDROGRAPH in m^3/s


.105+))))))*))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3)))))))))),
 D       *     *     .           .           .           .           .
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 i       *    *  *   .           .           .           .           .
*
 s       *        *  .           .           .           .           .
*
 c   .0753   *       .           .           .           .           .
3
 h       *         * .           .           .           .           .
*
 a       *         * .           .           .           .           .
*
 r       *           .           .           .           .           .
*
 g       *   *      *.           .           .           .           .
*
 e   .0503           .           .           .           .           .
3
         *           *           .           .           .           .
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 i       *  *        .*          .           .           .           .
*
 n       *           .*          .           .           .           .
*
         *           . *         .           .           .           .
*
 m   .0253 *         .  *        .           .           .           .
3
 ^       *           .   *       .           .           .           .
*
 3       **          .    **     .           .           .           .
*
 /       *           .           .           .           .           .
*
 s       **          .           .           .           .           .
*

.000*)))))))))))3)))))******3**********3**********3**********3**********
       4.          50.        100.        150.        200.        250.
289.

                                   Time, t (min)
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
4.	.002	64.	.029	124.	.000	184.	.000	244.	.000
7.	.008	67.	.025	127.	.000	187.	.000	247.	.000
10.	.017	70.	.022	130.	.000	190.	.000	250.	.000
13.	.029	73.	.019	133.	.000	193.	.000	253.	.000
16.	.045	76.	.016	136.	.000	196.	.000	256.	.000
19.	.063	79.	.000	139.	.000	199.	.000	259.	.000
22.	.081	82.	.000	142.	.000	202.	.000	262.	.000
25.	.094	85.	.000	145.	.000	205.	.000	265.	.000
28.	.102	88.	.000	148.	.000	208.	.000	268.	.000
31.	.105	91.	.000	151.	.000	211.	.000	271.	.000
34.	.103	94.	.000	154.	.000	214.	.000	274.	.000
37.	.097	97.	.000	157.	.000	217.	.000	277.	.000
40.	.089	100.	.000	160.	.000	220.	.000	280.	.000
43.	.079	103.	.000	163.	.000	223.	.000	283.	.000
46.	.070	106.	.000	166.	.000	226.	.000	286.	.000
49.	.059	109.	.000	169.	.000	229.	.000	289.	.000
52.	.052	112.	.000	172.	.000	232.	.000	0.	.000
55.	.044	115.	.000	175.	.000	235.	.000	0.	.000
58.	.039	118.	.000	178.	.000	238.	.000	0.	.000
61.	.034	121.	.000	181.	.000	241.	.000	0.	.000

```
+++ System hydrograph registers full at time =   289. min
  INL 6  7  0  106
  EFF 0  100  2.83  100
  PIP 28.95  153.76  153.00  151.84  151.26  -750
+++ Link #  7,  Flow depth =    .390 m, Critical depth =    .546 m
  PNC 3  2  1.22  48  0
  REM LINK 2 TO 1
  UHY HYDR02.QT
+++ Notice:  Intermediate file has data in SI units
  GUT 22.86  153.46  153.00  0.013  50   0.914  40  0  1
+++ Depth =   .028 m, Vel =    .746 m/s,  Width =   1.160 m
+++ Discharge =     .012 m^3/s, Slope =   .020 m/m
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

GUTTER HYDROGRAPH in m^3/s

```
.012+)))))**))))3)))))))))))3)))))))))))3)))))))))))3)))))))))))3))))))))),
 D       *       *   .           .           .           .           .
*
 i       *    *      .           .           .           .           .
*
 s   .0103        *  .           .           .           .           .
3
 c       *   *       .           .           .           .           .
*
 h       *         * .           .           .           .           .
*
 a       *         * .           .           .           .           .
*
 r   .0073           .           .           .           .           .
3
 g       *   *      *.           .           .           .           .
*
 e       *           .           .           .           .           .
*
         *           *           .           .           .           .
*
 i   .0053  *        .*          .           .           .           .
3
 n       *           .*          .           .           .           .
*
         *           . **        .           .           .           .
*
 m       * *         .  *        .           .           .           .
*
 ^   .0023           .   *       .           .           .           .
3
 3       **          .     **    .           .           .           .
*
 /       *           .           .           .           .           .
*
 s       **          .           .           .           .           .
*

.000*)))))))))))3)))))******3**********3**********3**********3**********
      4.          50.         100.        150.        200.        250.
289.
```

Time, t (min)

Example Four: Hydrographic Analysis and Pressure Flow Simulation

PLOT-DATA (Time, t(min) vs. Discharge, (m^3/s))

t	Q	t	Q	t	Q	t	Q	t	Q
4.	.000	64.	.003	124.	.000	184.	.000	244.	.000
7.	.001	67.	.003	127.	.000	187.	.000	247.	.000
10.	.002	70.	.002	130.	.000	190.	.000	250.	.000
13.	.003	73.	.002	133.	.000	193.	.000	253.	.000
16.	.005	76.	.002	136.	.000	196.	.000	256.	.000
19.	.007	79.	.000	139.	.000	199.	.000	259.	.000
22.	.009	82.	.000	142.	.000	202.	.000	262.	.000
25.	.010	85.	.000	145.	.000	205.	.000	265.	.000
28.	.011	88.	.000	148.	.000	208.	.000	268.	.000
31.	.012	91.	.000	151.	.000	211.	.000	271.	.000
34.	.011	94.	.000	154.	.000	214.	.000	274.	.000
37.	.011	97.	.000	157.	.000	217.	.000	277.	.000
40.	.010	100.	.000	160.	.000	220.	.000	280.	.000
43.	.009	103.	.000	163.	.000	223.	.000	283.	.000
46.	.008	106.	.000	166.	.000	226.	.000	286.	.000
49.	.007	109.	.000	169.	.000	229.	.000	289.	.000
52.	.006	112.	.000	172.	.000	232.	.000	0.	.000
55.	.005	115.	.000	175.	.000	235.	.000	0.	.000
58.	.004	118.	.000	178.	.000	238.	.000	0.	.000
61.	.004	121.	.000	181.	.000	241.	.000	0.	.000

```
+++ System hydrograph registers full at time =   289. min
  INL 7  7  0  107  0 0 0 0 0 0 1
  EFF 0  100  2.83  100
  PIP 7.62  153.00  152.39  151.11  150.81  -600
+++ Link #  8,  Flow depth =    .372 m, Critical depth =    .553 m
  PNC 2  1  0  0.0  2
  PFS 0  60  3.0  5  0  9  8  30  0.05
  PHJ 8  7  6  15  5  4  3  2  1
  PFP 1  2   3  4  5  6   7  8
  IQV 2.0  5.1  0  0  0  0  2.5  6.4
  IDY IDY 0.0  0.0  0.0  0.0  0.6  0.0  0.0  0.0
  END
 END OF INPUT DATA.
```

Example Four: Hydrographic Analysis and Pressure Flow Simulation

*** LINK 8 TO 5 Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	Diam (mm)
1	27	300	165.160	.233	1.250	.925	3.381	.080	29		
			158.760		1.250	.925	3.905	.276			
2	53	450	158.580	.058	1.430	.942	3.463	.184	29		
			155.500		1.430	.942	4.001	.636			
3	52	450	155.500	.058	1.430	.942	3.462	.184	29		
			152.510		1.410	.923	3.999	.636			

Length = 133. m Total length = 133. m

*** MAINLINE 15 TO 1 Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	Diam (mm)
4	23	300	154.190	.066	1.250	.925	2.972	.080	35		
			152.670		1.250	.925	3.276	.232			
5	28	450	152.480	.010	1.440	.953	3.561	.566	219	.308	508
			152.210		1.400	.913	1.625	.258			
6	49	750	151.900	.001	1.960	1.148	1.516	.670	185	.307	737
			151.840		1.920	1.107	.821	.363			
7	29	750	151.840	.020	1.920	1.107	3.360	.773	53		
			151.260		1.740	.928	3.312	1.463			
8	8	600	151.110	.039	1.890	1.240	4.320	.784	69		
			150.810		1.580	.930	4.001	1.131			

Length = 137. m Total length = 269. m

Example Four: Hydrographic Analysis and Pressure Flow Simulation

Hydraulic Gradeline Computations

Link #	Down-stream Node #	Hydraulic Gradeline Elevation	Crown Elev.	Possible Surcharge	Ground Elev.	Super-crit.?	Manhole Depth	Loss Coef
1	7	158.980	159.065	N	160.010	Y	.849	1.53
2	6	155.802	155.957	N	156.930	Y	.848	2.11
3	5	152.812	152.967	N	153.920	Y	2.882	2.65
4	5	152.890	152.975	N	153.920	Y	2.882	2.44
5	4	153.645	152.667	Y	153.860	N/A	1.198	5.74
6	3	152.940	152.602	Y	153.760	N/A	.999	1.23
7	2	151.806	152.022	N	153.000	Y	1.474	1.75
8	1	151.182	151.420	N	152.390	Y	.372	.00

Link #	Terminal Node #	Hydraulic Gradeline Elevation	Ground Elevation	Loss Coef.
1	8	166.145	166.410	1.50
4	15	154.989	155.440	1.50

Example Four: Hydrographic Analysis and Pressure Flow Simulation

Pressurized Flow Simulations

```
Total Simulation time:          60 min
Incremental time:                6 min
Length of integration step:      2 s
Initial time:                  .00 min
Surcharge variables: itMax      30 iterations
                     surTol   .050

Printed output at the following  9 junctions

      8      7      6     15      5      4      3      2      1

          and for the following  8 pipes

      1      2      3      4      5      6      7      8
```

Pipe Characteristics

Pipe Number	Length (m)	Area (m^2)	Manning "n"	Max. Width (m)	Depth (m)	Junctions at Ends		Invert Height above Junctions	
1	27.432	.073	.022	.305	.305	8	7	.000	.180
2	53.340	.164	.014	.457	.457	7	6	.000	.000
3	51.816	.164	.014	.457	.457	6	5	.000	.030
4	22.860	.073	.014	.305	.305	15	5	.000	.190
5	28.346	.164	.014	.457	.457	5	4	.000	.310
6	48.768	.456	.014	.762	.762	4	3	.000	.000
7	28.956	.456	.014	.762	.762	3	2	.000	.150
8	7.620	.292	.014	.610	.610	2	1	.000	.000

Junction Characteristics

Junction Number	Ground Elev.	Crown Elev.	Invert Elev.	Connecting Pipes	
8	166.410	165.465	165.160	1	
7	160.010	159.065	158.580	1	2
6	156.930	155.957	155.500	2	3
15	155.440	154.495	154.190	4	

Example Four: Hydrographic Analysis and Pressure Flow Simulation

Junction Characteristics

Junction Number	Ground Elev.	Crown Elev.	Invert Elev.	Connecting Pipes		
5	153.920	152.975	152.480	3	4	5
4	153.860	152.667	151.900	5	6	
3	153.760	152.602	151.840	6	7	
2	153.000	152.022	151.110	7	8	
1	152.390	151.420	150.810	8		

+++ Outfall control water-surface elev = 150.810 m

Summary of initial heads, flows and velocities

Pipe Number	Flow (m^3/s)	Velocity (m/s)	Pipe Number	Flow (m^3/s)	Velocity (m/s)	Pipe Number	Flow (m^3/s)	Velocity (m/s)
1	70.6	5.1	2	.0	.0	3	.0	.0
4	88.3	6.4	5	.0	.0	6	.0	.0
7	.0	.0	8	.0	.0	90009	.0	.0

Summary of initial depths

Junction Number	Depth (m)	Junction Number	Depth (m)	Junction Number	Depth (m)	Junction Number	Depth (m)
8	.000	7	.000	6	.000	15	.000
5	.000	4	.000	3	.000	2	.000
1	.000						

Example Four: Hydrographic Analysis and Pressure Flow Simulation

Summary of Junction Results

Junction Number	Ground /Invert Elev. (m)	Uppermost Pipe crown Elev. (m)	Maximum Computed Water Surface Elev	Time of Occurrence (h:min)	Surcharge at Max. Depth	Length of Surcharge (min)
8	166.410 165.160	165.465	165.270	0 : 33	.000	.000
7	160.010 158.580	159.065	158.745	0 : 30	.000	.000
6	156.930 155.500	155.957	155.665	0 : 30	.000	.000
15	155.440 154.190	154.495	154.311	0 : 33	.000	.000
5	153.920 152.480	152.975	153.920	0 : 22	.945	32.300
4	153.860 151.900	152.667	153.303	0 : 28	.636	27.767
3	153.760 151.840	152.602	153.366	0 : 25	.764	27.567
2	153.000 151.110	152.022	153.000	0 : 20	.978	31.633
1	152.390 150.810	151.420	150.810	0 : 0	.000	.000

Example Four: Hydrographic Analysis and Pressure Flow Simulation

Summary of Pipe Results

Pipe Number	Design Flo/Vel m^3/s|m/s	Pipe Vertical Depth (mm)	Maximum Computed Flo/Vel m^3/s|m/s	Time of Occurrence (h:min)	Ratio of Max. to Design Flow	Max. Depth above Invert Pipe Ends Up (m)	Dn (m)
1	.288 3.946	305.	.080 1.995	0 : 33	.3	.110	-.015
2	.664 4.043	458.	.185 3.461	0 : 30	.3	.165	.165
3	.664 4.041	458.	.185 1.352	0 : 30	.3	.165	1.410
4	.242 3.311	305.	.080 1.463	0 : 33	.3	.121	1.250
5	.270 1.642	458.	.503 3.062	0 : 24	1.9	1.440	1.093
6	.378 .829	763.	.612 1.642	0 : 26	1.6	1.403	1.526
7	1.527 3.347	763.	.824 2.282	0 : 21	.5	1.526	1.740
8	1.180 4.044	610.	1.315 7.105	0 : 20	1.1	1.890	.000

NORMAL END OF HYDRA

Example Five: Sanitary Sewer Design

Problem:

This example demonstrates the use of HYDRA in the design mode. A sanitary sewer is planned in which each individual in the service area contributes 378 L/day to the sewer (GPC 100) and such that infiltration into the system occurs at a rate of 9 353 L/day/ha. The exception is the area serviced by the Kenyon Street lateral, where there is a rate of 18 705 L/day/ha. Figure 17 shows the example site.

Peaking factor data are expressed in the PEA command. This command provides ordered pairs of flow versus peaking factor for flows between 3 x 10^{-4} and 283 m^3/s.

Other commands shown in the example include the PDA command (pipe data), which gives pipe characteristics, such as Manning's "n" value, and the CST, EXC, PCO, and TSL commands, which work together in making cost estimates for the preliminary design. These costs include trench construction, as well as pipe materials.

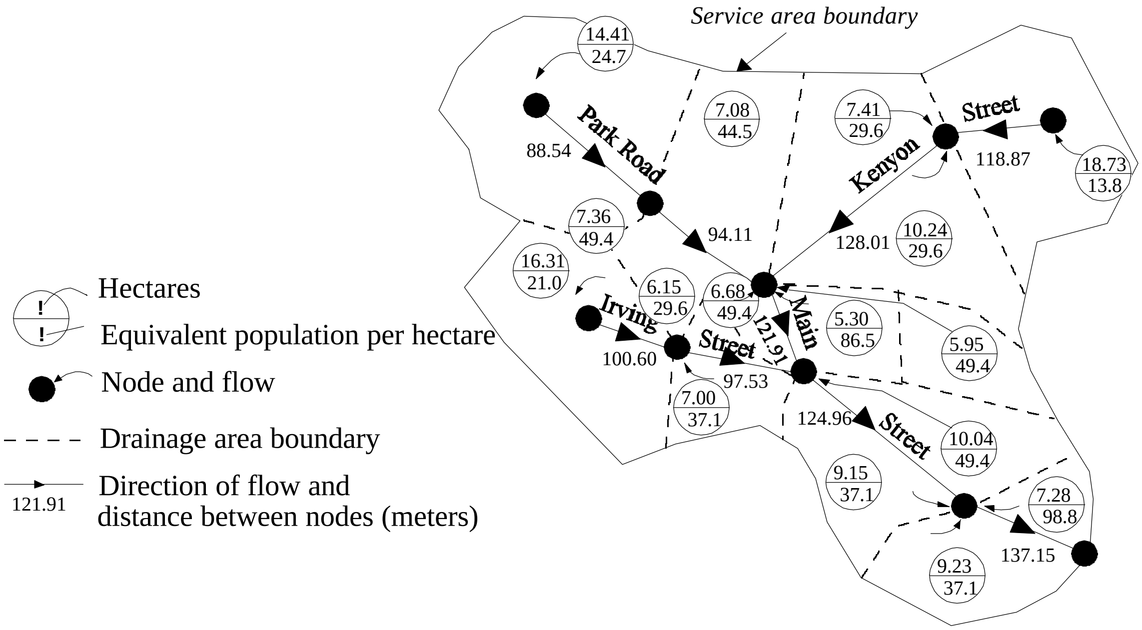

Figure 17. Sanitary sewer design.

Input file: HYDRA5.HDA

```
JOB Example Five: Sanitary Sewer Design
SWI 1
GPC 378.5
PEA 0.0003,4.46  0.0014,3.78  0.0028,3.3  0.0283,2.6  0.2831,2.1  2.831,1.7
    28.31,1.4  283.1,1.13
INF 0 9352.6
CST 1.5 0.457 0 0 0.152 0 0.65 0.5 0 5.23 1.15 0.52 3.27 4.58 2.69
EXC 0 0.98 3.05 0.98
PCO 8 8.20 10 11.48
PDA 0.013 150 2.13 1.22 0.762 0.001
TSL 0 0.2 3.05 0.2
NEW PARK ROAD
SAN 14.41 24.7
PIP 88.54 30.72 28.53
SAN 7.08 44.5
SAN 7.36 49.4
PIP 94.11 28.53 25.82
HOL 1
NEW IRVING STREET
SAN 16.31 21.0
PIP 100.60 28.95 24.75
SAN 6.15 29.6
SAN 7.00 37.1
PIP 97.53 24.75 22.65
HOL 2
NEW KENYON STREET
INF 0 18705.2
SAN 18.73 13.8
PIP 118.87 29.72 27.13
SAN 45.22 29.6
SAN 10.24 29.6
PIP 128.01 27.13 25.82
HOL 3
NEW MAIN STREET
INF 0 9352.6
REC 1
REC 3
SAN 6.68 49.4
SAN 5.30 86.5
SAN 5.95 49.4
PIP 121.91 25.82 22.65
REC 2
SAN 10.04 49.4
PIP 124.96 22.65 20.42
SAN 9.15 37.1
SAN 7.28 98.8
SAN 9.23 37.1
PIP 137.15 20.42 19.84
END
```

Discussion of output:

The results show that a 152-mm pipe has adequate capacity for the lateral links but larger diameter pipes are designated for the mainline links progressively.

Output file: HYDRA5.LST

```
        ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
                                                              Page No   1

   Example Five: Sanitary Sewer Design

 +++ Commands Read From File C:\HYDRA\HYDRA5.HDA
   JOB
   SWI 1
   GPC 378.5
   PEA 0.0003,4.46  0.0014,3.78  0.0028,3.3  0.0283,2.6  0.2831,2.1  2.831,1.7
    28.31,1.4  283.1,1.13
   INF 0 9352.6
   CST 1.5 0.457 0 0 0.152 0 0.65 0.5 0 5.23 1.15 0.52 3.27 4.58 2.69
   EXC 0 0.98 3.05 0.98
   PCO 8 8.20 10 11.48
   PDA 0.013 150 2.13 1.22 0.762 0.001
   TSL 0 0.2 3.05 0.2
   NEW PARK ROAD
   SAN 14.41 24.7
   PIP 88.54 30.72 28.53
 +++ Top width of trench ---> UP  1.1 m
                         ---> DN  1.1 m
 +++ Link #  1,  Flow depth =    .049 m, Critical depth =    .068 m
   SAN 7.08 44.5
   SAN 7.36 49.4
   PIP 94.11 28.53 25.82
 *** WARNING: Pipe invert at D/S end dropped to meet minimum depth
     to invert criterion at D/S end
 +++ Top width of trench ---> UP  1.1 m
                         ---> DN  1.1 m
 +++ Link #  2,  Flow depth =    .078 m, Critical depth =    .109 m
   HOL 1
   NEW IRVING STREET
   SAN 16.31 21.0
   PIP 100.60 28.95 24.75
 +++ Top width of trench ---> UP  1.1 m
                         ---> DN  1.1 m
 +++ Link #  3,  Flow depth =    .043 m, Critical depth =    .068 m
   SAN 6.15 29.6
   SAN 7.00 37.1
   PIP 97.53 24.75 22.65
 *** WARNING: Pipe invert at D/S end dropped to meet minimum depth
     to invert criterion at D/S end
 +++ Top width of trench ---> UP  1.1 m
                         ---> DN  1.1 m
 +++ Link #  4,  Flow depth =    .076 m, Critical depth =    .099 m
   HOL 2
   NEW KENYON STREET
   INF 0 18705.2
   SAN 18.73 13.8
   PIP 118.87 29.72 27.13
 +++ Top width of trench ---> UP  1.1 m
```

```
  Example Five: Sanitary Sewer Design

                        ---> DN  1.1 m
+++ Link #  5,  Flow depth =    .058 m, Critical depth =    .077 m
  SAN 45.22 29.6
  SAN 10.24 29.6
  PIP 128.01 27.13 25.82
*** WARNING: Pipe invert at D/S end dropped to meet minimum depth
    to invert criterion at D/S end
+++ Top width of trench ---> UP  1.2 m
                        ---> DN  1.2 m
+++ Link #  6,  Flow depth =    .123 m, Critical depth =    .141 m
  HOL 3
  NEW MAIN STREET
  INF 0 9352.6
  REC 1
  REC 3
  SAN 6.68 49.4
  SAN 5.30 86.5
  SAN 5.95 49.4
  PIP 121.91 25.82 22.65
*** WARNING: Pipe invert at D/S end dropped to meet minimum depth
    to invert criterion at D/S end
+++ Top width of trench ---> UP  1.2 m
                        ---> DN  1.2 m
+++ Link #  7,  Flow depth =    .126 m, Critical depth =    .186 m
  REC 2
  SAN 10.04 49.4
  PIP 124.96 22.65 20.42
*** WARNING: Pipe invert at D/S end dropped to meet minimum depth
    to invert criterion at D/S end
+++ Top width of trench ---> UP  1.2 m
                        ---> DN  1.2 m
+++ Link #  8,  Flow depth =    .162 m, Critical depth =    .210 m
  SAN 9.15 37.1
  SAN 7.28 98.8
  SAN 9.23 37.1
  PIP 137.15 20.42 19.84
*** WARNING: Pipe invert at D/S end dropped to meet minimum depth
    to invert criterion at D/S end
+++ Top width of trench ---> UP  1.3 m
                        ---> DN  1.3 m
+++ Link #  9,  Flow depth =    .247 m, Critical depth =    .215 m
  END
 END OF INPUT DATA.
```

Example Five: Sanitary Sewer Design

*** PARK ROAD Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
1	89	150	28.590	.025	2.130	1.968	1.102	.006	23854.
			26.400		2.130		1.355	.024	
2	94	150	26.400	.029	2.130	1.968	1.484	.014	25355.
			23.690		2.130		1.463	.026	

Length = 183. m Total length = 183. m
Cost = 49209. Total Cost = 49209.

*** IRVING STREET Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
3	101	150	26.820	.042	2.130	1.968	1.333	.006	27104.
			22.620		2.130		1.761	.031	
4	98	150	22.620	.022	2.130	1.968	1.274	.011	26276.
			20.520		2.130		1.265	.022	

Length = 198. m Total length = 198. m
Cost = 53380. Total Cost = 53380.

Example Five: Sanitary Sewer Design

*** KENYON STREET Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
5	119	150	27.590 25.000	.022	2.130 2.130	1.968	1.124 1.272	.007 .022	32026.
6	128	300	25.000 23.690	.010	2.130 2.130	1.805	1.256 1.384	.034 .098	66527.

Length = 247. m Total length = 247. m
Cost = 98553. Total Cost = 98553.

*** MAIN STREET Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
7	122	300	23.690 20.520	.026	2.130 2.130	1.805	2.032 2.206	.057 .156	63357.
8	125	300	20.520 18.290	.018	2.130 2.130	1.805	1.879 1.828	.072 .129	64942.
9	137	375	18.290 17.710	.004	2.130 2.130	1.724	1.133 1.032	.086 .114	88429.

Length = 384. m Total length = 1012. m
Cost = 216728. Total Cost = 417871.

Example Five: Sanitary Sewer Design

NORMAL END OF HYDRA

Example Six: Sanitary Sewer Analysis

Problem:

For this example, the system suggested by HYDRA in example five is entered in the command string as an existing system and analyzed. (Note the extra parameters on the PIP commands.) If no other changes are made, the same flows would be calculated, but the costing data are ignored. However, to fully demonstrate the review features of HYDRA, additional flow is added to the system by adding to the person/ac numbers in the SAN commands and adding 0.099 m^3/s through a FLO command on Main Street. (The FLO command might represent the discharge of water from an industrial facility.)

Input file: HYDRA6.HDA

```
JOB Example Six: Sanitary Sewer Analysis
SWI 1
GPC 378.5
PEA 0.0003 4.46 0.0014 3.78 0.0028 3.3 0.0283 2.6 0.2831 2.1 100 2.831
    28.31 1.4 283.1 1.13
INF 0 1531.79
CST 1.5 4.92 0 0 1.64 0 3.27 0.5 0 5.23 1.15 0.52 3.27 4.58 2.69
EXC 0 0.98 3.05 0.98
PCO 8 8.20 10 11.48
PDA 0.013 150 2.13 1.22 0.76 0.001
TSL 0 0.2 3.05 0.2
NEW PARK ROAD
SAN 14.41 32.1
PIP 88.54 30.72 28.53 28.59 26.39 -150
SAN 7.08 69.2
SAN 7.37 74.13
PIP 94.12 28.53 25.82 26.39 23.68 -150
HOL 1
NEW IRVING STREET
SAN 16.31 28.4
PIP 100.5 28.95 24.75 26.82 22.62 -150
SAN 6.15 37.1
SAN 7.00 44.5
PIP 97.53 24.75 22.65 22.62 20.51 -150
HOL 2
NEW KENYON STREET
INF 0 9352.6
SAN 18.74 21.2
PIP 118.87 29.72 27.12 27.58 24.99 -150
SAN 7.41 37.1
SAN 10.24 37.1
PIP 128.01 27.13 25.82 24.99 23.68 -300
HOL 3
NEW MAIN STREET
INF 0 9352.6
FLO 0.0991
REC 1
REC 3
SAN 6.68 56.8
SAN 5.30 93.9
SAN 5.95 56.8
PIP 121.91 25.82 22.65 23.68 20.51 -300
REC 2
```

```
SAN 10.04 56.8
PIP 124.96 22.64 20.42 20.51 18.29 -300
SAN 9.15 44.5
SAN 7.28 106.3
SAN 9.23 44.5
PIP 137.15 20.42 19.84 18.29 17.71 -450
END
```

Discussion of output:

The output shows that the sewer system is still able to adequately pass the flows on Park Road, Irving Street, and Kenyon Street, but portions of the Main Street lateral are overloaded. In these cases, HYDRA suggests remedies. For the 124.96-m link 8 under Main Street, HYDRA suggests somehow removing 0.027 m^3/s or constructing an additional 279-mm pipe to pass the flow.

Output file: HYDRA6.LST

```
          ******  HYDRA  ******* (Version 6.1) *****         Date 07-21-1998
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   Example Six: Sanitary Sewer Analysis

 +++ Commands Read From File C:\HYDRA\HYDRA6.HDA
   JOB
   SWI 1
   GPC 378.5
   PEA 0.0003 4.46 0.0014 3.78 0.0028 3.3 0.0283 2.6 0.2831 2.1 100 2.831
    28.31 1.4 283.1 1.13
   INF 0 1531.79
   CST 1.5 4.92 0 0 1.64 0 3.27 0.5 0 5.23 1.15 0.52 3.27 4.58 2.69
   EXC 0 0.98 3.05 0.98
   PCO 8 8.20 10 11.48
   PDA 0.013 150 2.13 1.22 0.76 0.001
   TSL 0 0.2 3.05 0.2
   NEW PARK ROAD
   SAN 14.41 32.1
   PIP 88.54 30.72 28.53 28.59 26.39 -150
 +++ Link #  1,  Flow depth =    .049 m, Critical depth =    .066 m
   SAN 7.08 69.2
   SAN 7.37 74.13
   PIP 94.12 28.53 25.82 26.39 23.68 -150
 +++ Link #  2,  Flow depth =    .082 m, Critical depth =    .113 m
   HOL 1
   NEW IRVING STREET
   SAN 16.31 28.4
   PIP 100.5 28.95 24.75 26.82 22.62 -150
 +++ Link #  3,  Flow depth =    .043 m, Critical depth =    .067 m
   SAN 6.15 37.1
   SAN 7.00 44.5
   PIP 97.53 24.75 22.65 22.62 20.51 -150
 +++ Link #  4,  Flow depth =    .075 m, Critical depth =    .097 m
   HOL 2
   NEW KENYON STREET
   INF 0 9352.6
   SAN 18.74 21.2
```

```
  PIP 118.87 29.72 27.12 27.58 24.99 -150
+++ Link #  5,  Flow depth =    .055 m, Critical depth =    .073 m
  SAN 7.41 37.1
  SAN 10.24 37.1
  PIP 128.01 27.13 25.82 24.99 23.68 -300
+++ Link #  6,  Flow depth =    .081 m, Critical depth =    .091 m
  HOL 3
  NEW MAIN STREET
  INF 0 9352.6
  FLO 0.0991
  REC 1
  REC 3
  SAN 6.68 56.8
  SAN 5.30 93.9
```

```
  Example Six: Sanitary Sewer Analysis


  SAN 5.95 56.8
  PIP 121.91 25.82 22.65 23.68 20.51 -300
+++ Link #  7,  Flow depth =    .225 m, Critical depth =    .278 m
  REC 2
  SAN 10.04 56.8
  PIP 124.96 22.64 20.42 20.51 18.29 -300
+++ Link #  8,  Flow depth =    .300 m, Critical depth =    .300 m
  SAN 9.15 44.5
  SAN 7.28 106.3
  SAN 9.23 44.5
  PIP 137.15 20.42 19.84 18.29 17.71 -450
+++ Link #  9,  Flow depth =    .346 m, Critical depth =    .293 m
  END
 END OF INPUT DATA.
```

```
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                                                            Page No   3

 Example Six: Sanitary Sewer Analysis

 ***  PARK ROAD                                  Analysis of Existing Pipes

                Invert        Depth Cover Velocity --Flow--      -Solutions-
 Link Length Diam  Up/Dn  Slope Up/Dn Up/Dn Act/Full Act/Full Load Remove Diam
        (m)  (mm)   (m)  (m/m)   (m)   (m)  (m/s)   (m^3/s)  (%) (m^3/s) (mm)
-------------------------------------------------------------------------------


   1    89   150  28.590   .025 2.130 1.968  1.094      .005   22
                  26.390        2.140 1.978  1.359      .024

   2    94   150  26.390   .029 2.140 1.978  1.513      .015   58
                  23.680        2.140 1.977  1.462      .026

-------------------------------------------------------------------------------
             Length =    183. m   Total length =    183. m


 ***  IRVING STREET                          Analysis of Existing Pipes

                Invert        Depth Cover Velocity --Flow--      -Solutions-
 Link Length Diam  Up/Dn  Slope Up/Dn Up/Dn Act/Full Act/Full Load Remove Diam
        (m)  (mm)   (m)  (m/m)   (m)   (m)  (m/s)   (m^3/s)  (%) (m^3/s) (mm)
-------------------------------------------------------------------------------


   3   101   150  26.820   .042 2.130 1.967  1.318      .005   17
                  22.620        2.130 1.968  1.762      .031

   4    98   150  22.620   .022 2.130 1.968  1.258      .011   48
                  20.510        2.140 1.977  1.268      .022

-------------------------------------------------------------------------------
             Length =    198. m   Total length =    198. m
```

Example Six: Sanitary Sewer Analysis

*** KENYON STREET   Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	Diam (mm)
5	119	150	27.580	.022	2.140	1.978	1.098	.006	29		
			24.990		2.130	1.968	1.272	.022			
6	128	300	24.990	.010	2.140	1.815	.995	.015	15		
			23.680		2.140	1.815	1.384	.098			

Length = 247. m   Total length = 247. m

*** MAIN STREET   Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	Diam (mm)
7	122	300	23.680	.026	2.140	1.815	2.493	.140	90		
			20.510		2.140	1.815	2.206	.156			
8	125	300	20.510	.018	2.130	1.805	2.211	.156	121	.027	279
			18.290		2.130	1.805	1.824	.129			
9	137	450	18.290	.004	2.130	1.642	1.325	.174	94		
			17.710		2.130	1.642	1.166	.185			

Length = 384. m   Total length = 1012. m

Example Six: Sanitary Sewer Analysis

NORMAL END OF HYDRA

Example Seven: Combined System Analysis

Problem:

HYDRA may be used to design or analyze a combined sanitary and storm sewer system. The example shown in figure 18 illustrates an analysis of a small drainage and service area feeding into a lateral under Hitda Road. No new commands are required beyond those used in separate analyses of storm drains or sanitary sewers. The hydrographic technique may also be used for combined sewer analysis and/or design.

Input file: HYDRA7.HDA

```
JOB Example Seven: Combined System Analysis
SWI 4
PEA 0.0004 3.71 0.0006 3.54 0.0008 3.45 0.0015 3.28  0.0028 3.1 0.0142 2.67
    0.0283 2.5 0.1416 2.15 0.2831 2.01 1.416 1.73 2.831 1.62
RAI 0 39.37 5 39.37 8 30.48 10 27.94 15 22.86 18 20.32 24 17.78 32 15.24
INF 0 4676.3
GPC 3785
PDA 0.013 200 1.22 0.914 0.610 0.001
EXC 1.52 0.95 7.62 1.50
TSL 0.914 0.5 3.05 0.5
PCO 200 8.83 900 81.50
CST 1.5 4.92 0 0 0 0.152 8.12 1 0 1.99 1.05 0.78 2.04 1.99 0
NEW HITDA ROAD
SAN 53.83 67.9
STO 53.83 0.25 30
PIP 152.39 30.48 30.48 29.08 28.89 -450
SAN 20.24 42.0
STO 20.24 0.2 25
SAN 8.50 30.9
STO 8.50 0.9 20
PIP 152.39 30.48 30.48 28.89 28.71 -450
SAN 16.19 21.0
STO 16.19 0.3 22
SAN 8.90 21.0
STO 8.90 0.3 18
PIP 30.48 30.48 30.02 28.71 28.65 -525
END
```

Discussion of output:

The results indicate that the entire system has the capacity to accommodate the respective flows as can be seen by the percentage of capacity noted in the output.

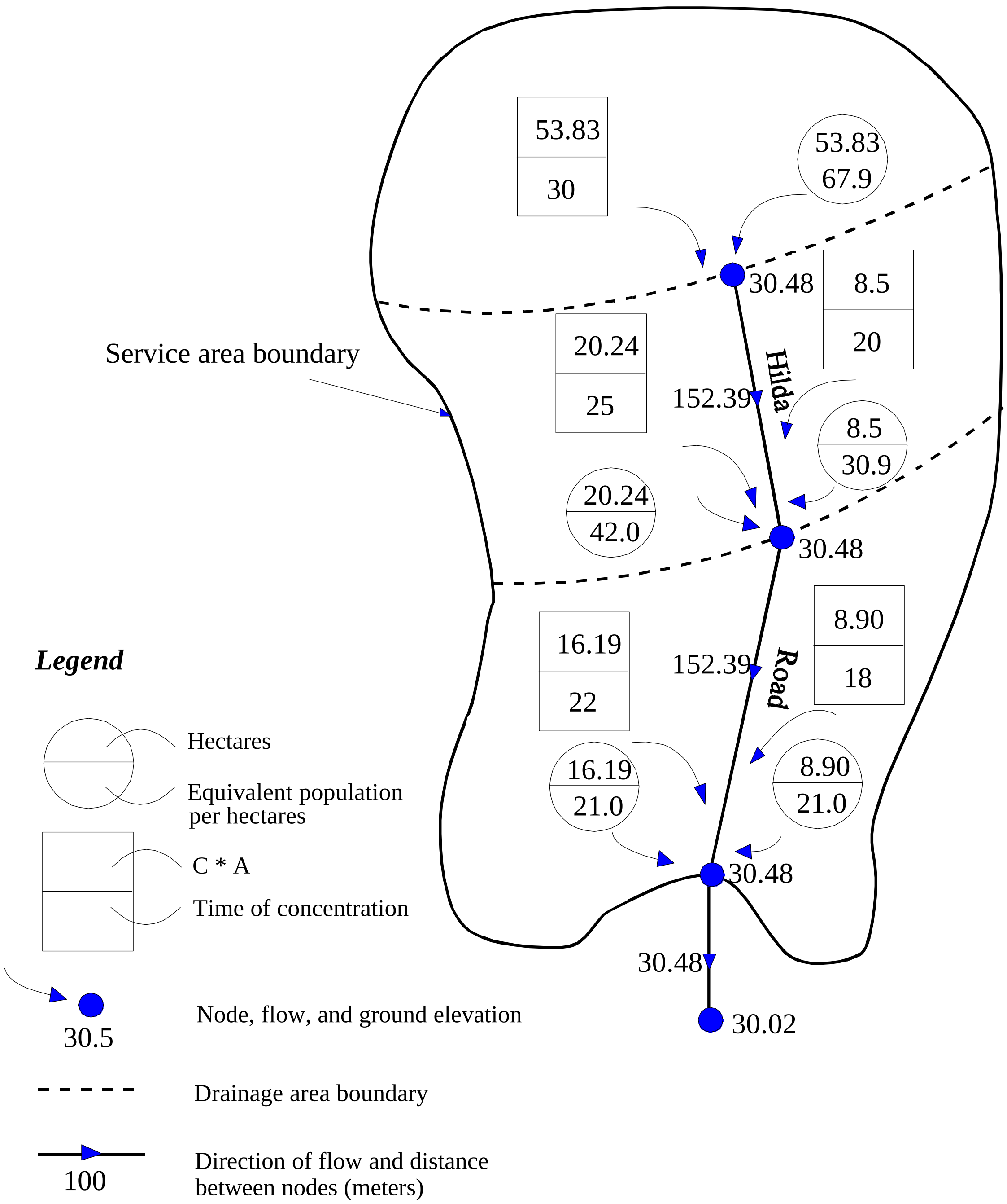

Figure 18. Combined system analysis.

Output file: HYDRA7.LST

```
          ******  HYDRA  ******* (Version 6.1) *****          Date 07-21-1998
                                                                 Page No   1

   Example Seven: Combined System Analysis

 +++ Commands Read From File C:\HYDRA\HYDRA7.HDA
   JOB
   SWI 4
   PEA 0.0004 3.71 0.0006 3.54 0.0008 3.45 0.0015 3.28  0.0028 3.1 0.0142 2.67
    0.0283 2.5 0.1416 2.15 0.2831 2.01 1.416 1.73 2.831 1.62
   RAI 0 39.37 5 39.37 8 30.48 10 27.94 15 22.86 18 20.32 24 17.78 32 15.24
```

IDF CURVE

```
39.*))))))))))3))))))))))3))))))))))3))))))))))3))))))))))3))))))))))3))),
        *         .         .         .         .         .         .
*
 R      *         .         .         .         .         .         .
*
 a      *         .         .         .         .         .         .
*
 i   30.3         .      *  .         .         .         .         .
3
 n      *         .         .         .         .         .         .
*
 f      *         .         *         .         .         .         .
*
 a      *         .         .         .         .         .         .
*
 l      *         .         .         *         .         .         .
*
 l   20.3         .         .         .     *   .         .         .
3
        *         .         .         .         .        * .         .
*
 i      *         .         .         .         .         .         .
*
 n      *         .         .         .         .         .         .
*
        *         .         .         .         .         .         .
*
 m   10.3         .         .         .         .         .         .
3
 m      *         .         .         .         .         .         .
*
 /      *         .         .         .         .         .         .
*
 h      *         .         .         .         .         .         .
*
        *         .         .         .         .         .         .
*
```

```
0.*))))))))))3))))))))))3))))))))))3))))))))))3))))))))))3))))))))))3)))-
       0.          5.         10.         15.         20.         25.        3
32.
```

Duration, t (min)

PLOT-DATA (Time, t(min) vs. Intensity, i(mm/h))

t	i	t	i	t	i	t	i	t	i
0.	39.	15.	23.	0.	0.	0.	0.	0.	0.
5.	39.	18.	20.	0.	0.	0.	0.	0.	0.
8.	30.	24.	18.	0.	0.	0.	0.	0.	0.
10.	28.	32.	15.	0.	0.	0.	0.	0.	0.

```
INF 0 4676.3
GPC 3785
PDA 0.013 200 1.22 0.914 0.610 0.001
EXC 1.52 0.95 7.62 1.50
```

```
  Example Seven: Combined System Analysis

  TSL 0.914 0.5 3.05 0.5
  PCO 200 8.83 900 81.50
  CST 1.5 4.92 0 0 0 0.152 8.12 1 0 1.99 1.05 0.78 2.04 1.99 0
  NEW HILDA ROAD
  SAN 53.83 67.9
  STO 53.83 0.25 30
  PIP 152.39 30.48 30.48 29.08 28.89 -450
+++ Tc =  30.0 min
+++ CA =  13.5
+++ Link #  1,  Flow depth =    .450 m, Critical depth =    .450 m
  SAN 20.24 42.0
  STO 20.24 0.2 25
  SAN 8.50 30.9
  STO 8.50 0.9 20
  PIP 152.39 30.48 30.48 28.89 28.71 -450
+++ Tc =  31.8 min
+++ CA =  25.2
+++ Link #  2,  Flow depth =    .450 m, Critical depth =    .450 m
  SAN 16.19 21.0
  STO 16.19 0.3 22
  SAN 8.90 21.0
  STO 8.90 0.3 18
  PIP 30.48 30.48 30.02 28.71 28.65 -525
+++ Tc =  33.3 min
+++ CA =  32.7
+++ Link #  3,  Flow depth =    .525 m, Critical depth =    .525 m
  END
 END OF INPUT DATA.
```

Example Seven: Combined System Analysis

*** HILDA ROAD Analysis of Existing Pipes

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Cover Up/Dn (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Load (%)	-Solutions- Remove (m^3/s)	-Solutions- Diam (mm)
1	152	450	29.080	.001	1.400	.913	1.428	.227	226	.127	508
			28.890		1.590	1.103	.633	.101			
2	152	450	28.890	.001	1.590	1.103	1.692	.269	275	.171	584
			28.710		1.770	1.282	.616	.098			
3	30	525	28.710	.002	1.770	1.201	1.316	.285	149	.094	432
			28.650		1.370	.801	.881	.191			

Length = 335. m Total length = 335. m

```
  Example Seven: Combined System Analysis

NORMAL END OF HYDRA
```

APPENDIX B. FOOTPRINTS FOR TYPICAL APPLICATIONS

This section presents basic HYDRA applications represented by arrangements of command strings. These arrangements of command strings, or footprints, are provided for the following typical applications:

1. Rational method.
2. Rational method with hydraulic gradeline and pressure flow simulations.
3. Hydrographic method.
4. Reservoir or impoundment routing.

These footprints are contained in files on the HYDRAIN package diskettes. These files contain command lines with empty data fields for which the user can supply the appropriate data (the footprint files should be copied and renamed before any editing is done. The simple pipe network which the first three footprints represent is depicted in figure 19.

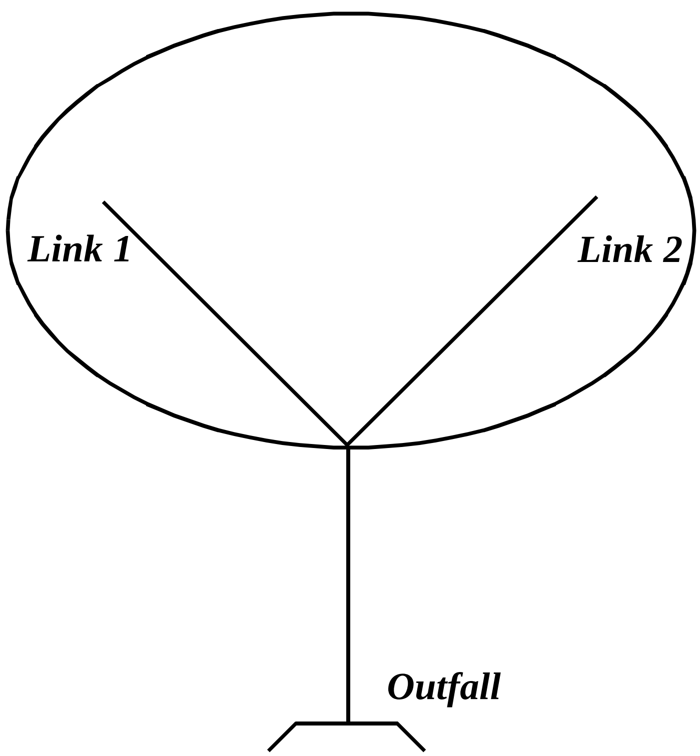

Figure 19. Schematic of footprint pipe network.

1. Rational Method Footprint

Filename: RTL˜FP.HDA

The following command string is typical for the Rational method design or analysis:

```
rem--->Rational Method Storm Drain Design/Analysis
rem--->REMOVE all unused command lines (Alt + D)
rem--->REQUIRED first command, JOB
JOB
SWI 2
RAI
PDA
rem--->OPTIONAL-Cost estimation---<CST>, <EXC>, <PCO>, <TSL> (design mode)
CST
EXC
PCO
TSL
rem--->OPTIONAL - additional cost commands---<ECF>, <LPC>, <PCF> (design mode)
ECF
LPC
PCF
rem--->Description of storm drain network: Link 1
NEW
STO
PIP
HOL
rem--->Description of storm drain network:  Link 2
NEW
STO
PIP
HOL
rem--->Description of storm drain network:  outfall
rem--->Recall system flows from Links 1 and 2 that were held:
NEW
REC
REC
PIP
END
```

2. Rational Method with Hydraulic Gradeline and Pressure Flow

Filename: HGLPF˜FP.HDA

The following command string is typical for rational analysis with hydraulic gradeline and pressure flow simulation:

```
rem--->Rational Method Storm Drain Analysis with HGL and Pressure Flow
Simulation
rem--->REMOVE all unused command lines (Alt + D)
rem--->REQUIRED first command, JOB
JOB
```

```
SWI 2
RAI
HGL
rem--->Description of storm drain network:  Link 1
NEW
STO
PIP
PNC
HOL
rem--->Description of storm drain network:  Link 2
NEW
STO
PIP
PNC
HOL
rem--->Description of storm drain network:  outfall
rem--->Recall flows in Links 1 and 2:
NEW
REC
REC
PIP
PNC
rem--->Pressure flow commands:  <TWE>, <PFS>, <PHJ>, <PFP>, <IQV>, <IDY>:
TWE
PFS
rem--->OPTIONAL-Print commands, <PFP>, <PHJ>:
PHJ
PFP
rem--->OPTIONAL-Initial condition commands, <IQV>, <IDY>:
IQV
IDY
END
```

3. Hydrographic Method

Filename: HYD˜FP.HDA

The following command string is typical for the hydrographic method (design or analysis):

```
rem--->Hydrographic Storm Drain Design/Analysis
rem--->REMOVE all unused command lines (Alt + D)
rem--->REQUIRED first command, JOB
JOB
SWI 3
STE
PDA
rem--->Description of storm drain network:  Link 1
rem--->GUT and INL commands must always precede the PIP command
NEW
UHY
GUT
INL
PIP
```

```
HOL
rem--->Description of storm drain network:  Link 2
NEW
UHY
GUT
INL
PIP
HOL
rem--->Description of storm drain network:  outfall
rem--->Recall flows from Links 1 and 2 that were held:
NEW
REC
REC
PIP
END
```

4. Reservoir or Impoundment Routing

Filename: DPON˜FP.HDA

The following command string is typical for routing system flows through some sort of storage facility.

```
rem--->Reservoir Routing of a Detention Facility
rem--->First command, JOB
JOB
SWI 3
rem--->Use time step as dictated by hydrograph
STE
rem--->Use Pipe DAta command to establish design criteria:
PDA
rem--->Run HYDRO to obtain user-specified hydrograph on
rem--->UHY command.
UHY
rem--->Commands that introduce hydrograph to the system, GUT and INL
GUT
INL
EFF
PIP
rem--->Stage-storage and performance (stage-discharge) curve represented
rem--->by SST and SDI, respectively:
SST
SDI
rem--->REServoir command must follow routing commands
RES    0,0
PIP
rem--->The last command of the data set, END
END
```

APPENDIX C: HYDRA COMMANDS

This appendix details the meaning and syntax of each command available in HYDRA. The descriptions are ordered alphabetically and include information on the command name, its purpose, and its structure. Any important notes pertaining to the command are also included.

COMMAND BEN - Pipe BENd data

Purpose: This command specifies the bend angle and radius for the computation of losses due to curved alignment of a pipe. This command should be placed after the PNC statement describing the pipe in which the bend occurs.

Structure:

BEN Rad, Angle

1) Rad - Bend radius of the link described by the previous PNC statement, m.
2) Angle - Bend angle of the link described by the previous PNC statement, deg.

Note: The bend angle must be between 0 to 115 deg, otherwise, HYDRA produces an error message.

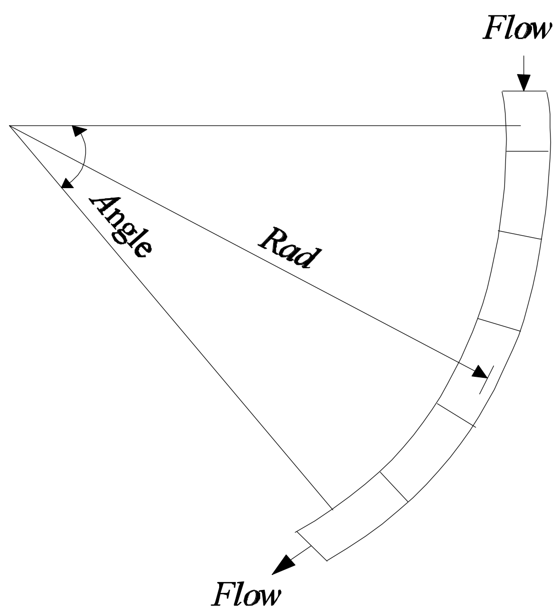

Figure 20. Description of BEN command.

COMMAND BOX - BOX Pipe

Purpose: This is one of the transport commands. It moves water from one point to another in a rectangular pipe. A PDA command is required somewhere in the command file ahead of the first BOX command.

Structure:

BOX length, grup, grdn, invup, invdn, minrise, span, lscost, trfac, plot

1) length - The length of the link or pipe, m. A negative number indicates a vertical box.
2) grup - Elevation of the ground at the upstream end of the link, m.
3) grdn - Elevation of the ground at the downstream end of the link, m.
4) invup - OPTIONAL - elevation of the invert of the pipe at the upstream end of the link, m.
5) invdn - OPTIONAL - elevation of the invert of the pipe at the downstream end of the link, m.
6) minrise - OPTIONAL - minimum depth of box, mm.
7) span - OPTIONAL - horizontal dimension of box, mm.
8) lscost - The lump sum to be added to the link cost calculated by HYDRA when cost parameters are used.
9) trfac - OPTIONAL - trench cost factor.
10) plot - OPTIONAL - enter 1 if plot is desired; enter 0 or leave blank if no plot is desired or if the Rational method is used.

Notes:

1) This command will accept 3, 5, 6, 7, 8, 9, or 10 parameters.
2) If “minrise” is given as a negative number, HYDRA will analyze the pipe assuming that the absolute value is its rise.

COMMAND CHA - CHAnnel

Purpose: CHAnnel is one of the HYDRA transport commands. It allows you to define an open channel or ditch.

Structure: (3 options)

CHA length, invup, invdn

---OR---

CHA length, invup, invdn, n

---OR---

CHA length, invup, invdn, n, lslope, bwidth, rslope (,eq, plot)

1) length - Length of link (distance between nodes), m.
2) invup - Invert elevation at the upstream end of the link, m.
3) invdn - Invert elevation at the downstream end of the link, m. If this value is less than 1.0 but greater than 0.0, HYDRA will take the value as a slope.

---OR---

4) n - Friction factor (Manning's "n").

---OR---

5) lslope - Slope of the channel's left side (horizontal : vertical).
6) bwidth - Width of the bottom (trapezoidal section), m.
7) rslope - Right side slope, (horizontal : vertical).
8) eq - (Optional) - enter 1 if gutter equation is to be used; enter 0 or leave blank if Manning's equation is to be used.
9) plot - (Optional) - enter 1 if plot is desired; enter 0 or leave blank if no plot is desired or if the Rational method is used..

Notes:

1) There is an automatic ditto feature in this command in that "n", left slope, bottom width, right slope, and eq will all be copied from the previous CHA command unless

overridden by subsequent entries in this command. The value of “n” may be changed without touching the geometry parameters, but all geometry parameters must be re-entered if one is. In other words, HYDRA will accept 3, 4, 7, 8, or 9 parameters for this command. Any other number of parameters is an error.

2) HYDRA does not check backwater problems on upstream channels or pipes so the designer should review the output carefully if such a condition is suspected.

3) The designer should consider the possibility for “scour” in channels where velocities are high.

4) The friction factor, “n”, has a major influence in the CHA calculation. Its selection should consider the channel future as well as existing condition.

COMMAND CRI - CRIteria

Purpose: **CRI** is a switch that indicates if the inverts or the crowns are matched in a “free design”.

Structure:

CRI switch1

switch1 - if set to zero, inverts will be matched in a free design; if set to 1, crowns will be matched.

Notes:

1) Initially, the switch is set to zero. In other words, if the command is not used inverts will be matched.

2) This command may be used at any location in the command string and any number of times.

COMMAND CST - CoSTs in place

Purpose: One of the required commands for a HYDRA cost estimate of pipe in place. CST sets several trench geometry factors and unit prices for material and haul.

Structure:

CST wmul, wadd, wmin, bmul, badd, bmin, $bed, pzmul, pzadd, $pz, shrink, $back, $waste, $borrow, $surf

1) wmul - Factor to establish trench width.
2) wadd - Added to product of wmul & interior pipe diameter, m.
3) wmin - Minimum trench width, m.
4) bmul - Factor to establish depth of bedding.
5) badd - Added to above to get depth of bedding, m.
6) bmin - Minimum depth of bedding, m.
7) $bed - Cost/$m^3$ of bedding in place.
8) pzmul - Factor to establish the depth of pipe zone material.
9) pzadd - Added to above to depth of pipe zone, m.
10) $pz - Cost/$m^3$ of pipe zone material in place.
11) shrink - Shrinkage of backfill material in place ($\geq$1.0).
12) $back - Cost/$m^3$ of replacing excavated backfill.
13) $waste - Cost/$m^3$ to remove excess native material.
14) $borrow - Cost/$m^3$ of borrow material in place.
15) $surf - Cost/$m^2$ for surface restoration.

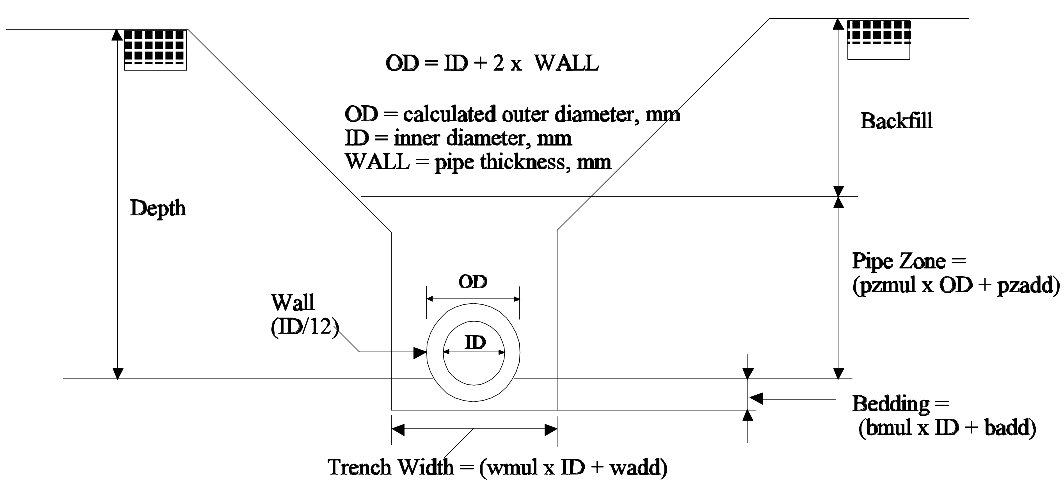

Figure 21. CST command trench and geometric factors.

COMMAND DIV - DIVert flow

Purpose: The primary function of this command is to model some form of overflow weir. It splits the system flow into two components - one that continues down the main system and another that is diverted in another direction.

Structure:

DIV hold, Qsys(1), Qdiv(1)...Qsys(n), Qdiv(n) (up to a maximum of 20 sets or a min of 2 sets - all values of Qsys must increase)

1)	hold	-	A HOLd number between 1 and 100 where DIVerted system flow is to be held. May be recalled with a REC command.
nx)	Qsys(i)	-	System flow, m^3/s.
ny)	Qdiv(i)	-	Flow, m^3/s, to be diverted. If the Qdiv exceeds Qsys then all of the system flow will be diverted, but not more than the system flow regardless of the value of Qdiv. If 0.0 then no flow is diverted.

Notes:

1) A maximum of 20 points (20 sets of Qsys and Qdiv) can be defined on the sys/div curve. All values of sys must increase div can be any value (so long as it is positive and not greater than sys).

2) This is a very flexible command and can be used for any type of weir or regulator in a system. It could be used to model a manhole that “blows its lid,” by loosing the diverted flow or by taking it back into the system at another point by using the REC or GET commands depending upon the situation. (The GET command is applicable only for hydrological analysis).

3) If this command is used without the SWI command or with the SWI command set to 1 or 2 HYDRA calculates the “peak” flow, looks at the DIV curve to find the amount to divert and then splits the population (in the event of sanitary flow) or Sum of C*A (in the case of normal storm flow) into two units. As a result the sum of the 2 downstream flows may be slightly greater than the upstream flow in some cases.

4) For hydrological analysis, both sanitary and misc. flow are dumped into the storm hydrograph so a rational split can be made. This will cause the sanitary flow to start over again and because of the peaking factor, result in the downstream flows being slightly greater than the upstream flow.

COMMAND ECF - Extra Cost per length

Purpose: An optional command, it allows costs that are related to depth to invert to be added to the pipe cost calculations. Examples of this are dewatering, sheeting, traffic control, etc.

Structure:

ECF depth(1), cost(1)...depth(n), cost(n) (up to a maximum of 20 sets or a minimum of 2 sets - all values of depth must increase)

nx)	depth(i)	-	Depth from the surface to the pipe invert, m.
ny)	cost(i)	-	Added cost/m of depth at the above depth.

Notes:

1) HYDRA interpolates between the points supplied for any values needed. For this reason, at least 2 sets of data must be supplied. The depth must always increase, the $/m can be any value.

2) More than one line may be used if needed.

COMMAND EFF - EFFiciency

Purpose: Ordered pairs of flow versus efficiency describing the performance of a user-supplied inlet. Specifies the performance characteristics of an inlet type other than those included in the INL command. (Must immediately follow INL command.)

Structure:

EFF flow(1), eff(1), flow(2), eff(2),... flow(n), eff(n) (up to a maximum of 20 pairs and a minimum of 2 pairs - all values of flow must increase)

nx)	flow(i)	-	Flow arriving at the inlet, m^3/s.
ny)	eff(i)	-	Capture efficiency corresponding to previous flow, expressed as a percentage.

COMMAND ELP - ELiptical Pipe

Purpose: This is one of the transport commands. It moves water from one point to another in a horizontal or vertical elliptical pipe. A PDA command is required somewhere in the command file ahead of the first ELP command.

Structure:

ELP length, grup, grdn, invup, invdn, minrise, span, lscost, trfac, plot

1)	length	-	The length of the link or pipe, m. A negative number indicates a vertical ellipse.
2)	grup	-	Elevation of the ground at the upstream end of the link, m.
3)	grdn	-	Elevation of the ground at the downstream end of the link, m.
4)	invup -		OPTIONAL - elevation of the invert of the pipe at the upstream end of the link, m.
5)	invdn -		OPTIONAL - elevation of the invert of the pipe at the downstream end of the link, m.
6)	minrise	-	OPTIONAL - minimum depth of ellipse, mm.
7)	span	-	OPTIONAL - horizontal dimension of ellipse, mm.
8)	lscost	-	The lump sum to be added to the link cost calculated by HYDRA when cost parameters are used.
9)	trfac	-	OPTIONAL - trench cost factor.
10)	plot	-	OPTIONAL - enter 1 if plot is desired; enter 0 or leave blank if no plot is desired or if the Rational method is used.

Notes:

1) This command will accept 3, 5, 6, 7, 8, 9, or 10 parameters.

2) If “minrise” is given as a negative number, HYDRA will analyze the pipe assuming that the absolute value is its rise.

COMMAND END - END of run

Purpose: This is the correct way to end a command string. When this command is encountered, HYDRA will end the run no matter how many commands follow.

Structure:

END (no data)

COMMAND EXC - EXCavation costs

Purpose: One of the required commands for a HYDRA cost estimate of pipe in place. It establishes trench excavation cost.

Structure:

EXC depth(1), cost(1)...depth(n), cost(n) (up to a maximum of 20 sets or a minimum of 2 sets - all values of depth must increase)

nx)	depth(i)	-	Depth from surface to excavation depth, m.
ny)	cost(i)	-	The cost in dollars/m^3 at excavation depth.

Notes:

1) HYDRA interpolates between the points supplied for any values needed. For this reason at least 2 sets of data must be supplied. The cost can be any value.

2) More than one line may be used if needed.

3) HYDRA calculates the excavation costs for every 1-m strip. The cubic meters of every strip is calculated and then the excavation cost of that strip is determined from the data supplied.

COMMAND FLO - miscellaneous FLOw

Purpose: FLO command adds a constant flow to the system. (A negative value indicates removal from the system.)

Structure:

FLO qpk

qpk - The volume of water to be entered into or removed from the system, m^3/s. In the case of removing flow, the value would be negative.

Notes:

1) This command must be followed by a PIP command before a HOL command is used or the flow will not be taken into the system.

2) Some of the uses of this command:

   a) Known major sources of infiltration.

   b) Cooling water.

   c) Subdrain flow.

   d) Stream flow.

   e) A given design flow.

   f) Irrigation tap (negative).

COMMAND GET - GET gutter hydrograph or peak flow

Purpose: GET is very similar to the REC command in function except it retrieves a gutter hydrograph or peak flow from data storage that was put there by the PUT or INL command. It has no use in nonhydrographic analysis.

Structure:

GET number

number - A number between 101 and 200 representing the gutter hydrograph or peak flow that was put in data storage by the PUT or INL command.

Note: For a rational analysis with inlets, GET must follow the STO command for bypass flow to be added to local runoff.

COMMAND GPC - unit flow of wastewater

Purpose: This command allows you to set the liters per capita per day that HYDRA is to use in its flow calculation in the SAN and SUN commands.

Structure:

GPC uflow

uflow - Unit flow of wastewater, in liters per capita per day, generated. This value will be used in conjunction with the peaking factor curve (PEA) and population data supplied through the SAN command to calculate actual sanitary flow.

COMMAND GUT - GUTter

Purpose: This is one of the transport commands and is available for storm drain analysis using the hydrographic technique, and to a limited extent, for Rational method analysis. In the hydrographic analysis, it allows the user to transport the hydrograph "overland," in a gutter, before merging it into the main system hydrograph. For the Rational method analysis, it provides spread, velocity and depth for the inlet command.

Structure:

```
              -- Uniform Gutter --
GUT  length, invup, invdn, n, Sx, bwidth (, 0, 0, plot)
                       or
              -- Composite Gutter --
GUT  length, invup, invdn, n, Sx, bwidth, Sw, 0 (, plot)
                       or
              -- Roadside Channel --
GUT  length, invup, invdn, n, lslope, bwidth, rslope, 1 (, plot)
```

1) length - Total length of the gutter, m.
2) invup - Invert elevation at the upstream end, m.
3) invdn - Invert elevation at the downstream end, m.
4) n - Manning's roughness coefficient for the gutter.
5) Sx or lslope - Roadway cross slope (uniform or composite gutter) or left side slope (roadside channel) horizontal distance per unit vertical (__:1).
6) bwidth - Width of the bottom of the trapezoidal channel bottom OR width corresponding to Sw for a composite gutter, OR width corresponding to Sx for a uniform gutter, m.
7) Sw or rslope- Composite gutter cross slope or right side slope (roadside channel) horizontal distance per unit vertical (__:1).
8) eq - OPTIONAL - enter 1 if using a roadside channel, enter 0 or leave blank if using a composite or uniform gutter.
9) plot - OPTIONAL - enter 1 if a plot is desired; enter 0 or leave blank if no plot is desired or if Rational Method is used.

Notes:

1) There is an automatic "ditto" feature in this command. Values for Manning's n, Sx or left slope, bottom width, Sw or right slope, and the eighth parameter will all be copied from the previous GUT command unless overridden by entries in this command. While the value of Manning's n may be changed without entering the remaining parameters; all of the gutter geometry parameters must be entered if there is a change in one of them.

2) The roughness coefficient "n" has a major influence in the gutter calculations so it should be selected with care.

3) All slopes, "lslope," "rslope," "Sx," and "Sw," are in units of distance horizontal per unit distance vertical. Therefore, large values (e.g., > 10) for the slopes mentioned would mean mild slopes and small values (e.g., < 3) would represent steep slopes.

COMMAND HGL - Hydraulic Gradeline Computation Switch

Purpose: This command signals to HYDRA that gradeline computations should be made. It should be placed before any flow generation commands.

Structure:

HGL Swi

Swi - A value of 0 or 1 should be entered to control the HGL computations:
0 - bypasses the computations. This is the default value.
1 - directs HYDRA to perform the HGL computations.

Note: Omission of this command will cause the HGL computations to be bypassed.

COMMAND HOL - HOLd lateral

Purpose: Holds system flow at the lower end of a lateral. This flow can then be recalled into a trunk at the appropriate time with the REC command.

Structure:

HOL number

number - A register number from 1 to 100 in which lateral data is to be stored. The REC command can then be used to bring this data back into the system. A register can be re-used any number of times but previously stored data will be lost. This should be of no concern if a REC command has already retrieved the data.

Note: It is good practice to immediately follow a HOL command by a NEW command. It is possible to lose data if not done.

COMMAND IDY - Initial Depth

Purpose: Establishes the initial depth in a junction for pressure flow evaluation.

Structure:

IDY y(1), y(2), y(3),...., y(n) (up to a maximum of 20 values)

y(j) - Initial depth in junction, m.

Notes:

1) IQV must precede IDY

2) This command is ignored in the rational formula analysis.

3) The junction depth is defined as the water level in the junction above the lowest pipe invert connected to the junction, usually the outflow pipe. The flow depth in the outflow pipe can often be estimated as the initial junction depth.

4) The order of initial depth entries should correspond to respective pipes as they appear on the PIP commands preceding the PFS command. The order of pipe entries, in turn, corresponds to the identification of new junctions. That is, each new pipe described adds a new junction to the system at its upstream end.

5) If any depth value, y(j), is zero or unknown, the zero must be entered.

6) The number of junctions is always 1 greater than the number of pipes. Therefore, the last initial depth corresponds to the outfall depth relative to the outfall pipe invert.

COMMAND INF - INFiltration

Purpose: Provide infiltration flows based on population or area.

Structure:

INF lcd, lhd (Use either)

1) lcd - Liters per capita per day of ground water finding its way into the system.

2) lhd - Liters per hectare per day of ground water finding its way into the system.

Notes:

1) Normally one would use gcd <u>or</u> gad, but not both at the same time. The one not used would be set to 0.0. Some feel the gcd better reflects the number of side sewer connections (where much of the problem exists), while others prefer the gad. Remember this can be changed during a run to reflect changing ground water conditions.

2) Infiltration is only calculated on sanitary (SAN or SUN) data, not the storm (HYD or STO) data.

3) This is an important parameter in all “real” sanitary sewer systems. In numerous cases, infiltration into a sanitary sewer system exceeds the waste water flows. The value selected should be based on the assumption that the system has been in the ground for 50 years or more--not on new construction standards.

COMMAND INL - storm drain INLet

Purpose: Allows specification of inlet characteristics and/or performance. Adds a runoff hydrograph generated by a UHY command and transported by a GUT command to the system (as specified in the subsequent CHA or PIP command) modified by inlet performance.

Structure: (four options)

INL id, itype, subtype, store, w (, length, 0, 0, perim, area, plot) << Grate Inlet

- OR -

INL id, itype, 0, store, w (, length, a, wdepr, 0, 0, plot) << Curb Inlet

- OR -

INL id, itype, 0, store, h (, length, a, wdepr, 0, 0, plot) << Slotted Drain Inlet

- OR -

INL id, itype, 0, store << User-defined Inlet

1) id - Inlet identification number (a suggestion is to use the node number at the inlet location).

2) itype - One digit code, between 1 and 7, that defines the inlet type to be analyzed or designed.

   (1) Grate on grade.
   (2) Curb opening on grade.
   (3) Slotted drain on grade.
   (4) Grate in sump condition.
   (5) Curb opening in sump condition.
   (6) Slotted drain in sump condition.
   (7) User-defined (command EFF must follow immediately after INL command in this case). ON GRADE ONLY. May be used to simulate combination inlets.

3)	subtype	-	One digit code, between 0 and 7, used for grate inlets (i.e., for inlet types one and four). This parameter must be set to zero if considering the other five inlet types. (0) Subtype not applicable. (1) Parallel bar P-1-7/8 in (50 mm). (2) Narrow parallel bar P-1-1/8 in (30 mm). (3) Curved vane. (4) Tilt bar - 45 degrees. (5) Safety parallel bar P-1-7/8-4 in (50-4 mm). (6) Tilt bar - 30 degrees. (7) Reticuline.
4)	store	-	Storage register for excess flow for hydrograph ANALYSIS of inlets on grade. When used, it must be a number from 101 to 200. When using the Rational method or hydrograph DESIGN mode, set this parameter equal to zero.
5)	w or h	-	Width (w) of grate or slot opening OR height (h) of curb inlet, m.
6)	length	-	OPTIONAL - length of inlet, m. Must be 150 mm or greater.
7)	a	-	OPTIONAL - inlet depression for curb or slot inlet, m. Enter zero if inlet not depressed or if depression should be calculated from previous GUT command.
8)	wdepr	-	OPTIONAL - width of inlet depression for curb or slotted inlets, m. Enter zero if inlet not depressed or if depression width should be calculated from previous GUT command.
9)	perim	-	OPTIONAL - inlet perimeter, m. Used only for grate inlet in sump condition.
10)	area	-	OPTIONAL - area of orifice opening of the inlet, m^2. Used only for grate inlet in sump condition.
11)	plot	-	OPTIONAL - enter 1 if a plot is desired; enter 0 or leave blank if no plot is desired or if the Rational method is used.

Notes:

1) Grate length can be greater than 6 000 mm, although if this occurs, a message will be displayed.

2) Zeros should be used as place holders to skip past unneeded parameters.

3) To properly analyze or design an inlet, **all** flows reaching an inlet must first travel through a gutter (GUT command). When appropriate, a nominal gutter length at the correct slope can be used.

COMMAND IPU - Individuals Per Unit

Purpose: Establishes population per "sanitary unit" for SUN command.

Structure:

IPU ipu

ipu - Number of individuals per each sanitary unit (SUN).

COMMAND IQV - Initial flow (Q) and Velocity

Purpose: Specifies the initial discharge and velocity in a pipe for a pressure flow simulation.

Structure:

IQV q(1), v(1), q(2), v(2),...., q(n), v(n) (up to a maximum of 20 pair)

nx) q(i) - Initial discharge in pipe, m^3/s.

ny) v(i) - Initial velocity in pipe, m/s.

Notes:

1) Initial flows and velocities correspond to the respective pipes in the order they appear in the data set.

2) In the detailed output (ndetail = 2 in the PFS command), HYDRA will report flows for a conceptual pipe which is used to record flow volume leaving the system through the outlet and to permit flow continuity computations at the outlet.

COMMAND JOB - JOB title

Purpose: Initiates job and enters job title.

Structure:

JOB jobtitle

jobtitle - Up to 50 alphanumeric characters describing your job.

Note: This must be the first command and there may be only one JOB in any command file. It allows you to “load” a job title into HYDRA that will be printed on every page of output.

COMMAND LOS - Additional pipe LOSses

Purpose: This command allows for the input of pipe losses in addition to those determined by the sewer system configuration. Losses input through this command will be included in the hydraulic gradeline computations.

Structure:

LOS ploss

ploss - A value for losses, m, in addition to those calculated by the hydraulic gradeline computations, experienced by the pipe described by the previous PNC command.

Note: This loss applies only to the pipe on the previous PIP command and should model minor losses due to structures other than junctions and manholes.

COMMAND LPC - List the Pipe Costs

Purpose: This command calculates and prints cost of pipe in place in a tabular form. The costs calculated are the unit costs resulting from parameters that you define with CST, EXC, PCO, and TSL commands.

Structure:

LPC diam, start depth, end depth, increment

1) diam - Inside diameter of pipe, m.
2) start depth - Invert depth, m, at which cost estimate table is to start.
3) end depth - Invert depth, m, of last cost estimate.
4) increment - The number of m between each cost estimate. If left blank or set to 0, HYDRA will set to 0.3.

Note: All data used in this command must be in 0.3 m increments, for example 6.0 or 6.3 is acceptable.

COMMAND MAP - MAP scale

Purpose: The SAN, SUN, and STO commands all require entry of land segment areas in ha. The user may choose to enter a raw ratio instead with a negative sign preceding each value. HYDRA converts to ha using the value set by this command. Even if the MAP command is in the command file, the user may still choose to enter ha directly.

Structure:

MAP scale

scale - The map ratio (scale) of the map being used, (_ : 1).

Notes:

1) As with nearly all HYDRA commands, this command can be used any number of times in the command string. This allows the user to use any number of different maps on a single system.

2) The formula used to calculate ha is illustrated below. This formula is used anytime “areas” are negative. The negative value is first changed to a positive value and then:

$$\text{Area, ha} = (\text{area, mm}^2) \bullet (\text{scale})^2 / 10{,}000\ \text{m}^2/\text{ha})$$

COMMAND NEW - start NEW lateral

Purpose: This command is used to initiate calculation for a new lateral.

Structure:

NEW lateral name

lateral name- Up to 20 alphanumeric characters.

Notes:

1) This command starts off a new lateral. Any flow entering this lateral must follow this command in the command string.

2) After the first lateral, be sure to use a HOL command before a NEW command if you intend to recall the flow from a lateral back into the system. The NEW command clears any flows calculated, but not explicitly saved.

COMMAND PCF - Pipe Cost Factor

Purpose: An optional command for cost estimates. Its purpose is to increase cost of pipe in place to reflect the additional cost of higher class pipe as trench depth increases and/or extra cost for laying the pipe.

Structure:

PCF depth(1), factor(1)...depth(n), factor(n) (up to a maximum of 20 sets, or a minimum of 2 sets - all values of depth must increase).

nx) depth(i) - Depth from the surface to the pipe invert, m.
ny) factor(i) - Some factor which when multiplied by pipe cost from the PCO command will result in a reasonable adjustment for a class of pipe.

Notes:

1) HYDRA interpolates between the points supplied for any values needed. For this reason at least two sets of data must be supplied. The factor can be any value.

2) More than one line may be used if needed.

COMMAND PCO - Pipe Costs

Purpose: One of the required commands for cost estimates. It establishes the cost per m of pipe in place.

Structure:

PCO dia(1), cost(1)...dia(n), cost(n) (up to a maximum of 20 sets or a minimum of 2 sets - all values of dia must increase).

nx)	dia(i)	-	Inside diameter of pipe, m.
ny)	cost(i)	-	Cost, dollars/m, of pipe laid in place but not backfilled.

Notes:

1) HYDRA interpolates between the points supplied for any values needed. For this reason at least two sets of data must be supplied. The cost can be any value.

2) More than one line may be used if needed.

COMMAND PDA - Pipe DAta

Purpose: This command establishes design criteria for use with the PIP command.

Structure:

PDA n, mindia, mindepth, mincover, minvel, minslope, maxdia

1) n - Pipe friction factor (Manning's "n").
2) mindia - The minimum diameter of a pipe in a "free design," mm.
3) mindepth - In a free design, HYDRA will not place the invert shallower than this depth, m.
4) mincover - In a free design, HYDRA will not allow cover over the pipe to be less than this value, m.
5) minvel - In a free design, HYDRA will not allow the full flow velocity to drop below this value, m/s.
6) minslope - In a free design, HYDRA will not select a slope less than this value, m/m. This value MUST be greater than zero (0.0).
7) maxdia - OPTIONAL - If it is used, HYDRA will not select a pipe larger than this diameter, mm. This parameter should probably not be used for the first run on any design.

Note: PDA is usually placed near the beginning of a input data set. It MUST be used somewhere prior to the first PIP command. Any changes in design criteria (i.e., two different pipes having different roughness factors) can be accomplished by repeated use of this command.

COMMAND PEA - PEAking factor

Purpose: In order to calculate sanitary flow in a sewer system, HYDRA must be given a "peaking factor" curve. This peaking factor curve is used to translate average daily flow into peak daily flow. This command is used to supply HYDRA with that curve.

Structure:

PEA adf(1), pf(1), adf(2), pf(2)...adf(n), pf(n) (up to a maximum of 20 sets, or a minimum of 2 sets - all values of m^3/s must increase).

nx)	adf(i)	-	Accumulated average daily flow from SAN and SUN commands.
ny)	pf(i)	-	The peaking factor. Normally ranges between 1.0 and 4.0.

Note: The last average daily flow value should be greater than the maximum expected adf in the system, otherwise the interpolating features of the command may yield unexpected results. Also, experience indicates that the curve should start at zero, as initial flows in a system are often very small.

COMMAND PFP - Printed Flow Pipe

Purpose: Contains list of pipes for which flows and velocities are to be printed for a pressure flow simulation.

Structure:

PFP pipe(1), pipe(2), pipe(3),...., pipe(npprt) (up to a maximum fo 20 values)

pipe(i) - Pipe number for detailed printout.

Note: The number of pipes entered should correspond to npprt on the PFS command.

COMMAND PFS - Pressure Flow Simulation

Purpose: Specifies control parameters for the pressure flow simulation option and initiates the pressure flow simulation.

Structure:

PFS tzero, nstime, delt, nsdelt, ndetail, njprt, npprt, itmax, surtol

1) tzero - Start time of simulation, min.
2) nstime - Total simulation time, min.
3) delt - Incremental time to be used to calculate flows, s.
4) nsdelt - Printing interval between points in history table (whole number).
5) ndetail - Printout type: select
   - 0 - summary table
   - 1 - summary and time history tables
   - 2 - summary, time history tables, and detailed printout including each result.
6) njprt - Number of junctions for detailed printing of head output when print option is 1 or 2 (20 maximum). Used by PHJ command.
7) npprt - Number of pipes for detailed discharge printing when print option is 1 or 2 (20 maximum). Used with PFP command.
8) itmax - Maximum number of iterations to readjust head and flow of surcharged junctions.
9) surtol - Fraction of flow in surcharged area to be used as the tolerance for ending surcharge iterations.

Notes:

1) "nstime" should be equal to or greater than the longest base time of hydrographs in the system plus the travel time for the longest path.

2) "delt" is critical in terms of computing time and stability of the program. If a time step provided by the user violates the preset stability limit, the program will select an appropriate time step.

3) "itmax" and "surtol" control the accuracy of the solution in surcharged areas. Flows and heads in these areas are recalculated until the difference between inflow and outflow is less than the tolerance limit selected, or until the maximum number of iterations specified has been reached. Effective values for "itmax" and "surtol" have been found to be 30 and 0.05 (5 percent difference between inflows and outflows), respectively.

COMMAND PHJ - Print Heads at Junctions

Purpose: Contains list of individual junctions for which water depth and water surface elevations are to be printed for a pressure flow simulation.

Structure:

PHJ junction(1), junction(2),...., junction(njprt) (up to a maximum of 20 values)

junction(j) - Junction number for detailed printout.

Note: The number of junctions entered should correspond to "njprt" on the PFS command.

COMMAND PIP - Circular PIPe

Purpose: This is one of the transport commands. It moves water from one point to another in a circular pipe. A PDA command is required somewhere in the command file ahead of the first PIP command.

Structure:

PIP length, grup, grdn, invup, invdn, mindia, lscost, trfac, plot

1) length - The length of the link or pipe, m.
2) grup - Elevation of the ground at the upstream end of the link, m.
3) grdn - Elevation of the ground at the downstream end of the link, m.
4) invup - OPTIONAL - elevation of the invert of the pipe at the upstream end of the link, m.
5) invdn - OPTIONAL - elevation of the invert of the pipe at the downstream of the link, m.
6) mindia - OPTIONAL - minimum diameter of pipe, mm.
7) lscost - The lump sum to be added to the link cost calculated by HYDRA when cost parameters are used.
8) trfac - OPTIONAL - trench cost factor.
9) plot - OPTIONAL - enter 1 if plot is desired; enter 0 or leave blank if no plot is desired or if the Rational method is used.

Notes:

1) This command will accept 3, 5, 6, 7, 8, or 9 parameters.

2) If "mindia" is given as a negative number, HYDRA will analyze the pipe assuming that the absolute value is its diameter.

COMMAND PNC - Pipe-Node Connection

Purpose: This command specifies the connection of links and nodes for the computation of the hydraulic gradeline. Each PNC statement must immediately follow a PIP statement.

Structure:

PNC Unode, Dnode, Bdn, Angle, Nodtyp, Bench

1) Unode - Node number connecting the upstream end of the link specified by the previous PIP statement.
2) Dnode - Node number connecting the downstream end of the link specified by the previous PIP statement.
3) Bdn - The width, m, of the manhole at the downstream end of the link (or the width of Dnode). If junction is not a manhole or drop inlet, enter 0 as a place holder.
4) Angle - The angle, deg, between the link specified by Unode and Dnode above and the outflow pipe leaving Dnode (0 to 360 degrees).
5) Nodtyp - Code signifying type of node at the downstream end.
   - 0 - manhole
   - 1 - pipe junction
   - 2 - outfall
6) Bench - OPTIONAL - Used only with manhole junctions. Enter a value of 0, 1, 2, or 3 to signify the type of benching present at Dnode:
   - 0 - flat bench (Default).
   - 1 - 1/2 bench
   - 2 - full bench
   - 3 - improved bench

Notes:

1) Five or six parameters will be accepted.

2) Selection of Nodtyp and Bench determines how energy loss calculations are performed in a hydraulic gradeline analysis.

COMMAND PON - surface PONding

Purpose: This command allows surface ponding of flows UHY, GUT, and GET commands. Pond size can be designed or reviewed.

Structure:

PON cap, return

1) cap - Desired pond capacity, m^3. (Enter zero to design pond size.)
2) return - Maximum return discharge, m^3/s. (Enter zero to design maximum return discharge.)

Note: If inflow hydrograph has multiple peaks, Option 1 may compute a larger return flow than necessary. In this case use Option 2 in a trial and error solution until desired pond capacity is obtained.

COMMAND PSZ - Pipe SiZe

Purpose: To allow the user to define available pipe sizes to which the pipe design process will be limited.

Structure:

PSZ pipsiz(1), pipsiz(2), pipsiz(3),....,pipsiz(n) (up to a maximum of 40 values - all values of pipsiz must be in increasing order)

pipsiz(i) - Size of pipe diameter that is available for Design Mode, mm.

COMMAND PUM - PUMp

Purpose: Lifts hydraulic gradient the specified amount and sizes discharge pipe.

Structure:

PUM length, elevout, maxvel, dia, cost

1) length - The length of the discharge pipe, m.
2) elevout - Discharge invert elevation, m.
3) maxvel - Maximum velocity in the discharge pipe, m/s.
4) dia - Diameter, mm, either zero or a positive number. If zero, HYDRA will select a diameter. If a diameter is given, HYDRA will ignore maxvel.
5) cost - Lump sum cost of the pump station and the discharge pipe. If not used, will be set to zero.

Note: Many sanitary and combined collection systems require pumps to life the waste water to a higher elevation. This command is used to model this requirement.

COMMAND PUT - PUT gutter flow into storage

Purpose: This is one of the commands that can be used for by pass simulation. It is very similar to the HOL command in function except it stores away the gutter hydrograph and peak flows . This gutter hydrograph or peak flow is recalled by GET.

Structure:

PUT number

number - A number from 101 to 200 that labels the storage location. These storage locations are identical to those used by the INLet command, and can be re-used any number of times - however, if the same register is used more than once, the previous data will be destroyed. Recalling information from a PUT register can only be done with a GET command.

Note: Recalling information from a PUT register can only be done with a GET command. Although the GET command recalls the information stored in a PUT register, it does not destroy the data in the register. Therefore, if the user wishes to “see” the gutter hydrograph or peak flow at any point, he may use the PUT command to store it away, then the GET command to bring it back leaving an “image” of the hydrograph or peak flow in the PUT register.

COMMAND RAI - RAInfall data

Purpose: The rational formula requires an intensity-versus-duration curve. This command allows you to set the values on the curve. There are two formats for this command.

Structure:

RAI time(1), int(1)...time(n), int(n)...(up to a maximum of 20 sets or a minimum of 2 sets - all values of time must increase)

- OR -

RAI filename

nx)	time(i)	-	The duration of a specific rainfall intensity, min, which is experienced in a storm of a certain return period (such as a 5-year storm).
ny)	int(i)	-	Rainfall intensity, mm/h.
	filename	-	The filename of the externally produced IDF data set located in the intermediate directory and typically having the file extension "IDF."

Notes:

1) More than one line can be used if needed.

2) HYDRA interpolates between the points supplied for any values needed. For this reason, at least two sets of data must be supplied. The time must always increase; the intensity can be any value.

3) It is always wise to have the last two points have the same intensity, so the extension of the curve will never go negative.

4) Data in the file for the second command form must exist in the "ITM" subdirectory and must be in the following format (HYDRO uses this format):

Line 1:	Comment line 1	
Line 2:	NPTS	<< number of points
Line 3+:	DUR(n), INT(n)	<< duration, in min; intensity, mm/h

The data in lines three through the end must be in adjacent 10 space fields beginning in column 1. Make sure that no tabs exist in any of the fields in the file.

COMMAND REC - RECall lateral

Purpose: Recalls flow into the system that was stored using the HOL or DIV command.

Structure:

REC number

number - A number from 1 to 100 representing the storage location of data previously stored by a HOL or DIV command.

Notes:

1) CAUTION - HOL and DIV commands use the same 100 registers to store flow, so do not inadvertently overwrite them - however, HYDRA will warn of this problem.

2) There is a limit of five recalls at one node. If more than five laterals contribute to a downstream link, insert a short PIPe (say, 1.0 m) and then recall the rest.

COMMAND REM - REMarks

Purpose: Allows a line for remarks or comments.

Structure:

REM Any alphanumeric information.

Note: REM commands can be entered anywhere in the data set after the JOB command.

COMMAND RES - REServoir

Purpose: This command allows the user to analyze the effect of in-line storage on system flows. It is ignored on non-hydrographic runs.

Structure: (2 options)

-- OPTION 1 --

RES Cap, Return

1)	Cap	-	The capacity of the reservoir, m^3. If Cap is set to 0.0, HYDRA will select the required capacity and print the size immediately following the command. If you select the size and it is not sufficient, HYDRA will use what it can, bypass the excess, and give a message stating that the capacity was exceeded.
2)	Return	-	The maximum rate of return to the system from the reservoir, m^3/s. If the upstream system flow is less than this value, the reservoir is not used. If the upstream system flow ever exceeds this value, the reservoir will start to fill.

-- OPTION 2 --

RES 0, 0

Analysis using user-supplied stage-storage and stage-discharge curves as specified in the SST and SDI, respectively. Routing by the storage-indication method is performed.

1)	Cap	-	0 (zero value needed)
2)	Return	-	0 (zero value needed)

Notes:

1) Because this command must deal with a system hydrograph for its analysis, flows being carried in the INF, SAN and SUN registers are transferred to the storm hydrograph, and these registers cleared. This causes two minor problems that should not normally have a significant impact on the results.

2) Must be preceded by a transport command.

COMMAND SAF - SAFety factors

Purpose: Provides a safety factor to be applied to flows calculated by HYDRA.

Structure:

SAF san, inf, storm, flo

1)	san	-	Safety factor for sanitary flows generated by the SAN and SUN commands.
2)	inf	-	Safety factor for infiltration flow generated by the INF command working on data supplied by the SAN and SUN commands.
3)	storm	-	Safety factor for all flows generated by the STO or UHY commands.
4)	flo	-	Safety factor for flow generated by the FLO command.

Notes:

1) None of the parameters can be less than 1.00.

2) The above values should all be set to 1.00 when calibrating a system against recorded flows so HYDRA will not indicate an overloaded condition, when none exists.

3) This command may be used any number of times in the command string, so if you are analyzing a system in which some laterals are existing and some are being designed, altering the safety factors from 1.0 to design values should be considered.

4) If this command is not used, all values will be set to 1.0.

COMMAND SAN - SANitary flow

Purpose: Enters sanitary flow into the system. It uses data supplied in the PEA and GPC commands.

Structure:

SAN area, density

1)	area	-	Area, ha, served by the following link. The user can input map measurements, in^2, (as a negative value) using the MAP command to make the conversion to ha.
2)	density	-	The equivalent population per ha. This value normally ranges from 6 to 50.

Notes:

1) This command must be followed by a transport command (PIP, CHA, PUM, etc.) before a HOL command is used.

2) This command generates flow as follows:

   a) If area is negative (mm^2), the MAP command is used to convert the mm^2 to ha.

   b) Area is multiplied by parameter 2 to yield population; this is then added to upstream population.

   c) This total population is converted to m^3/s using criteria set in the GPC command.

   d) The Peaking (PEA) factor curve is used to adjust this flow for peak loads on the system.

   e) The area or population in this command is used in conjunction with the data supplied by the INF command to calculate the infiltration into the system.

COMMAND SDI - Stage-Discharge curve

Purpose: This command allows the user to input a stage-discharge pairs for the purpose of analyzing the effect of in-line storage on system flows. This command is intended for use with the RES and SST commands.

Structure:

SDI stage(1), dschrge(1), stage(2), dschrge(2),...,stage(n), dschrge(n) (up to a maximum of 20 pairs)

nx)	stage(i)	-	The stage of the reservoir, m.
ny)	dschrge(i)	-	The discharge of the reservoir corresponding to the stage specified above, m^3/s.

Notes:

1) This command must come before the RES command specifying the reservoir to which it applies. It will apply to all following reservoirs that are to be analyzed using the storage-indication routing methodology until a new SDI command is encountered.

2) If stage exceeds the last "stage" value, HYDRA calculates discharge based on a linear interpolation of the last two ordered pairs. The user should verify that this is appropriate when it occurs.

COMMAND SST - Stage-Storage curve

Purpose: This command allows the user to input stage-storage pairs for the purpose of analyzing the effect of in-line storage on system flows. This command is intended for use with the RES and SDI commands.

Structure:

SST Stage(1), Strge(1), Stage(2), Strge(2),...Stage(n), Strge(n) (up to a maximum of 20 pairs

1) Stage(i) - The stage of the reservoir, m.
2) Strge(i) - The storage of the reservoir corresponding to the stage specified above, m^3.

Notes:

1) This command must come before the RES command specifying the reservoir to which it applies. It will apply to all following reservoirs, that are to be analyzed using the storage-indication routing methodology, until a new SST command is encountered.

2) If stage exceeds the last “Stage” value, HYDRA calculates storage based on a linear interpolation of the last two ordered pairs. The user should verify that this is appropriate when it occurs.

COMMAND STE - STEp

Purpose: Sets the length of the step in hydrographic simulation.

Structure:

STE min/step

min/step - min/step. If this command is not used, the min/step defaults to 15 min. In this case, there will be 15 times 96 or 24 h total simulation.

Note: HYDRA works with 96 internal registers in hydrographic simulation. For this reason, when the step length is set to 5 min, the period of calculation is 8 h. When the step is set to 15 min, the period of calculation is 24 h, etc.

COMMAND STO - STOrm flow

Purpose: Enters subbasin data for use in the rational formula determination of storm water design flow. There are three options for this command, each allowing an additional level of complexity in determining the time of concentration.

Structure: (3 options)

STO area, C, time

-- OR --

STO area, C, oup, odn, odis, gtime

-- OR --

STO area, C, oup, odn, odis, gup, gdn, gdis

1) area - Service area, ha (or mm^2 if input with a negative value).
2) C - A value between 0.0 and 1.0 which represents the fraction of rainfall that runs off the watershed.

3)	time	-	Minimum time, min, runoff takes to get from the most hydraulically remote point of the subbasin to the inlet.

- or -

3)	oup	-	Upper overland flow elevation, m.
4)	odn	-	Lower overland flow elevation, m.
5)	odis	-	Overland flow distance, m (measured from oup to odn).
6)	gtime	-	Gutter time, min, (measured from odn to the inlet).

- or -

6)	gup	-	Upper gutter elevation, m.
7)	gdn	-	Lower gutter elevation, m.
8)	gdis	-	Length of the gutter, m (measured from odn to the inlet).

Notes: This command must contain 3, 6, or 8 parameters.

COMMAND SUN - Sanitary Units

Purpose: Enters the number of "sanitary units" contributing to the sanitary flow at a node.

Structure:

SUN units, area

1) units - The number of "units" (as in dwelling units) contributing to the system at this node.
2) area - OPTIONAL - The area containing the units, ha. If the MAP command has been used, mm^2 can be entered (as a negative value) and HYDRA will calculate the area.

Notes:

1) IPU, GPC and PEA commands must precede this command.

2) The set of commands, GPC, IPU, and SUN, can be included in the same link with the SAN command and HYDRA will assume that there are two contributing areas.

COMMAND SWI - criteria SWItch

Purpose: This command establishes the method by which HYDRA is to analyze storm and/or sanitary flows.

Structure:

SWI switch

switch - A number 1, 2, 3, 4, or 5, where:
- 1 - Sanitary only.
- 2 - Storm (Rational) only.
- 3 - Storm (Hydrographic) only.
- 4 - Sanitary and Rational.
- 5 - Sanitary and hydrographic.

COMMAND TRA - TRAnsfer system flow to surface flow

Purpose: This command enables system flow (pipe flow) to be added to surface flow (gutter flow).

Structure:

TRA (no parameters required)

Note: A HOL command cannot be used after TRA.

COMMAND TSL - Trench side SLope

Purpose: One of the required commands for HYDRA to make a cost estimate of pipe in place. It sets the slope of the trench side walls above the top of the pipe.

Structure:

TSL depth(1), slope(1)...depth(n), slope(n) (up to a maximum of 20 sets or a minimum of 2 sets - all values of depth must increase)

nx)	depth(i)	-	Depth from ground surface to the invert, m.
ny)	slope(i)	-	Slope of the portion of the trench above the top of the pipe when the invert is at the above depth, m/m.

Notes:

1) HYDRA interpolates between the points supplied for any values needed. For this reason, at least 2 sets of data must be supplied. The slope can be any value.

2) More than one line may be used if needed.

COMMAND TWE - TailWater Elevation

Purpose: This command allows for the input of a tailwater elevation at the system outfall.

Structure:

TWE elout

elout - The tailwater elevation at the system outfall, m.

COMMAND UHY - User Hydrograph

Purpose: Enables the user to use externally produced hydrographs in an analysis.

Structure: (2 options)

UHY time(1), flow(1), ... , time(n), flow(n) (up to a maximum of 96 sets or a minimum of 2 sets–all values of time must increase)

---OR---

UHY filename

(nx)time(i) - Time, min, of hydrograph.
(ny)flow(i) - Discharge, m^3/s, of hydrograph at time(i).
filename - The filename of the hydrograph data set located in the intermediate directory. The HYDRO produced hydrographs have the file extension "QT."

Notes:

1) Data in the file must exist in the "ITM" subdirectory and must be in the following format (HYDRO uses this format):

   Line 1 : Comment Line 1
   Line 2 : NPTS
   Line 3+: time (n), flow (n)

where:
NPTS = number of points
N = point number

2) The data in lines three through the end must be in adjacent 10 space fields beginning in column 1. Make sure that no tabs are entered in any of the fields in the hydrograph file.

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